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THE BEHAVIOR AND STRENGTH OF BOLTED SHINGLE SPLICES

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ABSTRACT

Bolted shingle joints are currently designed by methods developed for riveted joints in single shear which do not make effective use of the high shear strength of the fasteners. In this study, the results of a series of analytical studies and a complimentary test program examining the strength and behavior of bolted shingle joints are presented. Criteria for the design of shingle joints based on the observed behavior are suggested.

The variables examined in the analytical studies were (a) the A_n/A_s ratios, (b) the number of fasteners, (c) the number of fasteners per region, and (d) the number of regions. Similar to butt joints, the average shear strength of shingle joints was shown to decrease with joint length.

The test program was developed from the analytical studies to show experimentally that the predicted trends in joint behavior were valid. Nine shingle joints of A572 steel were tested. Two joints were fastened with 7/8 in. Huckbolts. The remaining joints were fastened with 7/8 in. A325 bolts. The slip characteristics and strength of the Huck-bolted joints were comparable to the behavior of the A325 bolted joints. The test results confirmed the indications of previous studies that the

slip in shingle joints tends to be less than the hole clearance. Various design methods approximating the distribution of load in the joint elements are compared with the experimental load partitions at the working load level. A preferred method of design is recommended.

1. INTRODUCTION

1.1 Background

Shingle joints, that is, joints in which the main components are spliced at various locations along the length of the joint, have been used extensively in heavy tension members to reduce the amount of splice material. These joints are designed by methods developed for riveted joints in single shear using various approximations to determine the distribution of plate forces and the shear transfer along the joint length. When high strength bolts were considered as a replacement for rivets in buildings, bridges and other steel structures, friction-type bolted joints were often used. These joints were considered comparable to riveted joints and did not take full advantage of the high shear strength of the bolts. It was believed that the methods used to proportion shingle joints and the assumption that they were slip critical, were unduly conservative and most likely wasteful.

1.2 Summary of Previous Study

Fisher and Yoshida summarized the previous experimental and theoretical work on large riveted bridge joints.[6] They reported on the testing of two large shingle joints which simulated part of a chord member and splice from the Baton Rouge Interstate Bridge. One large joint was fastened with A325 bolts and the other with A502 Gr. 1 rivets. The work was limited to an evaluation of joint behavior in the elastic range, up to and including major joint slip. The testing was terminated when the machine capacity was reached, thus, the ultimate strengths of the joints could not be determined.

The fastener design stress was taken as twice the value normally used for bridge joints. The force in a main plate was assumed to be transmitted to the lap plates in proportion to their distance from the main plate and fasteners were provided to resist these forces.

It was observed in the experiment that load was transferred from all main elements to the lap plates as these elements progressed into the joint. This resulted in substantially more load being carried by the lap plates than was assumed in the joint design. The forces in each discontinuous plate element were transferred primarily into the adjacent plate elements.

It was noted that large and complex bolted joints are unlikely to slip the full amount of the bolt-hole clearance. The slip (rigid-body movement) that occurred in the large bolted joint was about 50% of the hole clearance. Slip also occurred in the large riveted joint and was about 2/3 the amount observed in the bolted joint. The overall deformation of the large riveted joint always exceeded the comparable deformation in the large bolted joint at any given load level.

Since it was not possible to determine the ultimate strength of the two large simulated bridge joints, the net cross sectional areas of the joints were reduced so that failure could occur within the machine capacity. During the modification the ratio of net plate area to fastener shear area, A_n/A_s was maintained.

The results of the re-testing of these modified joints were reported by Rivera and Fisher.[9] The ultimate strength and load partition in the modified joints were determined experimentally. At the ultimate load of 3550 kips in the Modified Bolted Joint, there was considerable variation in the load carried by the individual fasteners. The study showed that the end fasteners in the first region were critical and that fasteners installed in the interior regions were not very effective. The Modified Riveted Joint provided a better redistribution of force to the interior regions due to the greater flexibility of the rivets. The bolted joint however, was 27% stronger than the riveted joint. The end fasteners of both joints failed by unbuttoning.

It was also indicated that the partition of load estimated by the design method was not very satisfactory. The method substantially over-predicted the critical shear transfer in the interior regions.

A theoretical load partition for shingle joints in the elastic range was also developed.[9] At that time, a mathematical model for shingle joints had not yet been developed for the inelastic case.

Desai and Fisher reported on the development of a mathematical model for shingle joints that permitted the complete force-displacement relationship to be predicted up to the ultimate load.[3] In the analysis, it was possible to specify the number of critical shear planes through each fastener. Assuming complete double shear, the ultimate strength of the Modified Bolted Joint and the Modified Riveted Joint were predicted within an accuracy of 3.9% and 8.5% respectively.

1.3 Objective of This Study

The previous experimental studies on the Simulated Bridge Joints[6] and the Modified Joints[9] were of exploratory nature. Further studies were required to evaluate in detail the ultimate strength characteristics of shingle joints so that the full range of behavior would be known.

In the present study, both analytical studies and a complimentary experimental program covering a wide range of parameters were undertaken. The local and overall deformation characteristics of shingle joints were further examined to confirm the previous observations that the expected slip in large shingle joints is less than the hole clearance. The behavior of the joints in the working load range and in the non-linear range was also examined, and existing methods for determining the approximate distribution of load were compared with the experimental load partitions. The test results were also intended to provide experimental confirmation for the theoretical solution for shingle joints suggested in Ref. 3 and to show the validity of the ultimate strength trends predicted in the analytical studies.

The final object was the development of design criteria that would provide the basis for specification provisions leading to more economical and safe design.

2. PRELIMINARY ANALYTICAL STUDIES

2.1 Introduction

The theoretical analysis for shingle joints developed in Ref. 3 was used to study analytically the effects of various joint geometries on the ultimate strength. The non-dimensionalized ratio of the predicted ultimate strength to the working load of the joint, P_u/P_w, was used as an index of joint behavior. Changes in the ratio resulting from variations in joint geometry were examined.

The idealized joints were assumed to have A572 steel plates fastened by A325 high strength bolts. The yield stress and ultimate tensile strength of the plates were taken to be 60 ksi and 88 ksi, respectively. The working loads for the joints were determined from the main plate net areas.

The variables studied were (a) the ratio A_n/A_s, defined as the ratio of the net main plate area in the first region to the total effective fastener shear area; (b) the total number of fasteners, N; (c) the number of fasteners per region; and (d) the number of regions.

2.2 Joint Behavior

Fig. 1 shows the change in joint strength with length for values of A_n/A_s ranging from 0.375 to 1.00 for shingle joints with three equal length regions. The fasteners were assumed to act in double shear in all three regions. This corresponds to a variety of allowable shear stresses. Ratios of A_n/A_s between 0.375 and 0.5 are typical of current friction-type joints. An A_n/A_s ratio of 0.625 corresponds to a shear stress of 22 ksi. As observed in previous studies of butt joints, a decrease in average joint strength occurred with an increase in length.[4,5]

Figure 1 indicates that only minor changes in joint strength beyond the working load resulted in spite of substantial variations in joint proportions. Joints with A_n/A_s ratios of 0.75 have 20% higher working loads than joints with A_n/A_s ratios of 0.625. The decrease in ratio P_u/P_w between the two A_n/A_s levels was less than 5%. A 40% increase in plate capacity between the 0.625 and 0.875 A_n/A_s ratio only resulted in a 10% decrease in the P_u/P_w ratio. This indicates that the same number of fasteners are capable of satisfactory behavior at allowable shear stresses up to 40% higher than used in current practice for bearing-type joints.

The strengths of the joints summarized in Fig. 1 were predicted assuming double shear behavior throughout the joint length. In Fig. 2, the effect of assuming only single shear behavior in the interior regions and its effect upon the predicted ultimate strengths is shown. The ratios, P_u/P_w, from Fig. 1 are compared with the predicted strengths based on the assumption of double shear in the first region and single shear in the interior regions.

At the lower A_n/A_s ratios, the predicted strengths of the joints were comparable for both idealizations of joint behavior. At higher A_n/A_s levels the load carried by interior fasteners was greater and a reduction in effective shear area had a more pronounced influence on joint strength.

Assuming a reduction in effective shear area increases the A_n/A_s ratios, and causes an increase in fastener stress. The 0.625 A_n/A_s ratio for double shear corresponds to an allowable shear stress of 22 ksi. With the interior fasteners in single shear this becomes 0.938. This corresponds to an effective shear stress of about 34 ksi at the main plate working load level. Hence the predicted strength of shingle joints with double shear in Region 1 and single shear in the interior regions at a 34 ksi stress level is about the same as shingle joints proportioned with the fasteners in double shear at a 22 ksi stress level. At lower A_n/A_s ratios, it is apparent that the assumption of either double shear or single shear in the interior regions did not affect the predicted ultimate strength significantly.

Figure 3 shows the computed fastener stress assuming double shear behavior in a shingle joint with an A_n/A_s ratio of 0.50. The load transferred to the lap plates in the first region was greater than the load transferred in the interior regions as indicated by the higher shear stresses. The fasteners in the interior regions were not as effective as those in Region 1. Figure 4 shows the predicted fastener stress in the same joint assuming single shear behavior in the interior regions. The predicted ultimate strength was unaffected since comparable behavior occurred in the first region which was critical. The amount of load transferred in each region was about the same as predicted for double shear. The stress in the interior regions was nearly doubled since only one shear plane was assumed to be effective. Corresponding to the reduction in effective shear area was an increase in A_n/A_s ratio from 0.50 to 0.75.

2.3 Variation in Region Length

A study was made to determine the effects of varying the number of fasteners in each region. The total number of fasteners and the plate areas were maintained, but the region lengths were adjusted by shifting an equal number of fasteners from each interior region into the first region. This is shown schematically in Fig. 5 along with the predicted variations in joint strength. Double shear behavior was assumed in Region 1 with single shear behavior in the interior regions.

In certain cases, fastener failure was predicted in the interior regions when the fasteners were rearranged. At the 0.75 A_n/A_s level, this occurrence was observed in the shorter joints when 4 fasteners were shifted into the first region. A slight decrease in strength was predicted. Essentially no variation in strength occurred in the longer joints.

At the 1.125 A_n/A_s level, slight increases in strength were predicted by shifting 2 fasteners into the first region. Shifting 4 fasteners caused interior fastener failures in the shorter joints, and thus, the increase in predicted strength was not substantial. The maximum predicted variation in strength was about 7% and was observed to decrease in the longer joints.

It was concluded that the predicted strength of shingle joints of a given length was not greatly influenced by rearranging the fasteners.

2.4 Number of Regions

A study was made to determine the effect of varying the number of main plate terminations, i.e., the number of regions in joints. In the study, the strengths of joints with one, two and three regions and the same number of fasteners were compared. Double shear behavior was assumed in the first region with single shear in the interior regions. The one-region joints were symmetrical butt joints having the total main plate area terminated at one location. In the two-region joints, the main plate area was terminated in equal amounts at two separate locations. The three-region joints had the geometry shown in Fig. 2.

Figure 6 shows the change in ratio, P_u/P_w, due to variation in the number of regions. At the 0.5 A_n/A_s ratio, little variation in strength was predicted by changing the number of regions. Similar results were shown in Fig. 2 for three-region joints when the assumptions of complete double shear and modified double shear were used. Compared to the butt joints with 0.75 A_n/A_s ratios, the two- and three-region joints were less efficient. Greater variation was predicted in the shorter lengths, however it is doubtful that short joints would be shingled.

At higher A_n/A_s ratios, the distribution of load in the interior fasteners is greater than at lower A_n/A_s ratios. Thus, terminating the main plates at different locations and thereby reducing the effective shear area causes a reduction in predicted ultimate strength.

3. DESCRIPTION OF TESTS

3.1 Test Program

A test program consisting of nine shingle joints was developed on the basis of the preliminary analytical studies. The program was intended to show experimentally that the ultimate strength trends predicted by the analytical studies were valid, and to further verify the theoretical solution for the strength of shingle joints.[3]

The geometry of the test joints is summarized in Tables 1 and 2. Each joint was composed of a single line of fasteners. For each joint, Table 1 gives the total number of fasteners and the number of fasteners in each region. Also listed are the joint lengths, the individual region lengths and the gage distances. In Table 2, the areas of the 1-inch main and lap plate components for each joint region are given. Joints 1, 2, 5, 7 and 8 were 3-region joints intended to show experimentally the effects of removing fasteners from the interior regions, and shifting fasteners from the interior regions into the first region. Joints 7 and 8 were comparable to a gage strip from the Modified Bolted Joint reported in Ref. 9. Joint 7 had twice as many fasteners as the Modified Bolted Joint, and

Joint 8 had twice the number in the first region. The joint areas were comparable as illustrated in Table 2.

Joints 3 and 4 were designed as 2-region joints intended to show the effect of the number of regions and variations in A_n/A_s ratio. Joint 3B was identical to 3A except for the type of fasteners. Huck fasteners were used in Joints 2 and 3B. All other joints were fastened with 7/8 in. A325 bolts.

3.2 Fabrication

All shop work for the fabrication of the plate assemblies was done at the American Bridge Company fabricating shops in Ambridge, Pennsylvania. The individual plate assemblies were cut from 2 large plate sections of the same rolling. A strip 3 feet wide was cut from the center of each large plate for material property tests. All holes were sub-drilled in the large plate sections prior to cutting. The individual plates were flame cut and then finished to the specified dimensions after assembly. All holes were then reamed to 15/16 in. In the gripping ends of the test joints, the plates were held in place with continuous 1/4 inch bead welds to insure a uniformity of wedge grip action during testing. The joints were shipped with temporary holding bolts.

Bolting-up operations were carried out at Fritz

Engineering Laboratory, Lehigh University. The turn-of-nut method of tightening was used for the joints fastened with A325 bolts. Bolt elongations were measured after tightening to determine the clamping forces.[10]

The Huckbolts were installed with equipment furnished by the Huck Manufacturing Company. Figures 7 and 8 describe schematically the Huckbolt and illustrate the installation process. A detailed description of the Huckbolt is given in Ref. 7.

3.3 Material Properties

The plates used for the experimental program were of ASTM A572 Grade 50 structural steel from the same heat. Material properties were determined from a series of standard plate coupon tests. The mean static yield was 49.0 ksi with a standard deviation of 2.25 ksi and the mean tensile strength was 79.0 ksi with a standard deviation of 3.12 ksi.

Three separate lots of A325 bolts were used. Bolts with a 5 in. grip were used in Joints 1, 3A, 4 and 5. Joint No. 6 required a special lot due to its 8 in. grip. Tension shear jig tests were conducted to determine the shear strength and ultimate shear deformation.[11] The calibration test results are summarized in Table 3. Load-deformation relationships for direct tension and torque tension were also developed for Lot SA bolts to be used in determining the bolt clamping force. These relationships were already available for bolt Lot XA.[8]

The bolts used in Joints 7 and 8 were from the same Lot used in the Modified Bolted Joint.[9] Since the grip distance in the Modified Bolted Joint was 4-1/2 inches, Lot G bolts, originally made for this grip, slightly underfit the 5 inch grip required in Joints 7 and 8. There was insufficient bolt extension outside the plates to engage the full thread of the nuts as observed in the sawed section of Joint 7 in Fig. 17. A recess of about 3/16 in. occurred at the ends of the bolts. Both shear and tension calibration tests were conducted using the 5 inch grip to determine its effect upon the shear strength and clamping force. The results are summarized in Table 3 and are compared to the results for the normal 4-1/2 inch grip with a full nut.[6,9] The bolts provided the same shear strength for both grip lengths since no shear plane intersected the threads. However, a 10% decrease in torque tensile strength was observed. The average clamping force at 1/2 turn was similarly affected as shown in Table 3. The average clamping force of 45.5 kips measured in Joints 7 and 8 still exceeded the specified minimum tension in spite of the lack of full nut engagement.

Shear calibration tests for the Huck fasteners were also conducted. The results are listed in Table 3.

3.4 Instrumentation and Test Procedure

The instrumentation of the test specimens was similar to that reported in Ref. 9. Fig. 9 illustrates a typical instrumentation set-up. Dial gages were positioned at locations of main plate termination and midway between to record local slip behavior. Overall elongation was measured by dial gages along both faces of each joint. Electrical resistance strain gages were used to determine the distribution of force in the main and lap plates along the length of the joint. Lines were also scribed along the plate edges at positions of bolt centerlines to show the amount of hole offset and relative plate movement.

All of the test joints were loaded to failure in static tension. A 5,000,000 lb. universal testing machine with flat wedge grips was used. Fig. 10 shows a specimen in the testing machine prior to loading. The procedure used was similar to earlier studies.[1,6,9] The joints were loaded in 100 kip increments in the elastic range. Dial gage readings were taken at each increment of load and as joint slip was experienced. The load increments were reduced to 50 kips as non-linear behavior was encountered and then to 25 kips as the predicted ultimate load was approached. Strain recordings were taken at all load increments until joint failure.

4. TEST RESULTS AND DISCUSSION

4.1 Slip Behavior

Figure 11 shows the load-deformation behavior of Joint 1 which was typical of the behavior observed in the other test joints. The shingle joints normally exhibited two separate load levels or stages at which major slip occurred. At the first slip load, substantial rigid body movement occurred along the shear plane adjacent to the main plate terminations with little or no movement along the second shear plane. The overall elongation at the first slip was about 50% of the total bolt-hole clearance, as shown in Fig. 11. At the second slip load, rigid body movement was experienced along the second shear plane with some additional slip occurring along the first shear plane. The total overall movement was normally less than the bolt-hole clearance. Each major slip was accompanied by a sudden decrease in the load applied by the testing machine, as shown in Fig. 11.

Table 4-A compares the overall slip behavior of the test joints. Listed are the joint clamping forces and the first and second major slip loads. Values of the slip coefficient, K_s, corresponding to the first major slip load are given, assuming (a) two equal shear planes and (b) double shear in the first

region and single shear in the interior regions (called effective slip in Col. 7).

It appears that assuming two equal shear planes when computing the slip coefficient for shingle joints can be misleading. Unlike butt joints, the transfer of load in shingle joints is not equal along each shear plane due to the unsymmetric positioning of plate terminations. This may lead to premature slip along one or more slip planes. (A relatively low slip coefficient could then be indicated for the joint by assuming an equal shear transfer). Slip coefficients below 0.3 were found for Joints 1, 4, 5, 7 and 8 assuming an equal shear transfer. The effective slip coefficients (Col. 7) computed on the assumption of double shear in the first region and single shear in the interior regions, are in better agreement with other test data.

The clamping forces used in the calculation of the slip coefficients for Joints 2 and 3B were determined from the mean clamping force found by installing several Huck fasteners in a calibrator. It is believed that the actual clamping forces developed by the fasteners in the joints are 10 to 15% higher than the forces indicated by the calibrator. An apparent increase in slip coefficient for these joints occurred as expected.

It was evident that the shorter, stiffer shingle joints with high A_n/A_s ratios, such as Joints 2, 3A and 3B, provided the greatest slip resistance. In the other joints, major slip first occurred at loads corresponding to somewhat lower slip coefficients.

The least slip resistance was observed to occur in Joint 7 where the slip coefficient, assuming double shear, was 0.147. In the previous large bolted shingle joint which had comparable plate area but half the number of fasteners, a slip coefficient of 0.31 was reported.[6] This was nearly twice the value found for Joint 7 indicating that the slip resistance was not increased by doubling the number of fasteners.

In Joint 7 slip only occurred along the shear plane adjacent to the plate terminations. No rigid body movement was observed along the other shear plane. As shown in Fig. 12, the total amount of slip was small, amounting to about 50% of the total bolt-hole clearance. This was comparable to the amount of slip observed in the earlier large bolted shingle joint.[6]

As reported in Ref. 6 and also found in this study, the total amount of slip in shingle joints tends to be less than the hole clearance. Since shingle joints are most often used where reversal of stress is unlikely because of the large dead loads, it appears reasonable to assume that shingle joints are not slip-critical. Hence, the emphasis in design could be placed on joint strength rather than slip.

Figure 13 shows the load-deformation behavior that occurred at two locations along Joint 1. As observed in earlier studies, two types of response were encountered.[6] The response shown in the top half of Fig. 13 was typical of the behavior observed at the ends of the joints and at main plate terminations. Elastic deformation between the main and lap plates was observed prior to the major rigid body movement experienced at slip. The second type of response observed in the joints is shown in the bottom portion. Near the center of the joints where no discontinuities occurred, the forces in adjacent plates were more nearly comparable. No relative movement was observed along the shear planes until major slip was experienced.

The amount of slip along the shear plane adjacent to the plate terminations was greater than the rigid movement along the other shear plane.

The magnitudes of the local slip at the ends of the plates were always larger than the slip indicated by the total elongation gages (compare Figs. 11 and 13). This condition was also observed previously.[7]

4.2 Joint Strength

The shingle joints tested in this series exhibited two distinct types of behavior in the non-linear range. Those with relatively high A_n/A_s ratios provided load-deformation curves with relatively little non-linear deformation, as shown in Figs. 11 and 14. This behavior was also typical for Joints 2, 5 and 6. Multiple bolt shear failures occurred in these joints. As shown in Fig. 15, all six bolts in the first region of Joint 1 were sheared. In Joint 5 (Fig. 16) all the fasteners in the interior regions were sheared. Complete shear failures were observed in Joints 2, 3A and 3B. The ultimate loads for all test joints are listed in Table 4B.

The second type of observed behavior was in joints with lower A_n/A_s ratios. The load-deformation curves were characterized by a long flat portion after gross section yielding, as shown in Fig. 12. This behavior was also typical of Joints 4 and 8. Failure occurred by either a shearing off of the end fastener accompanied by necking in the main plates or by fracture of the plates. In Joint 4, shown in Fig. 17, a fracture occurred at the bottom of the joint in the lap plates. Considerable necking was also evident in the main plate at the top of the joint.

Figure 18 shows the sawed section of Joint 8 illustrating the fastener deformation after failure. The end fastener had sheared off along the shear plane adjacent to the plate cutoffs. The amount of bolt deformation decreased rapidly from the end fastener toward the middle of the joint confirming that the end fasteners were critical. An apparent double-shear condition existed in the first 6 or 7 fasteners of Region 1, as indicated by the deformation along both shear planes. Thereafter, the fasteners appeared to be essentially in single-shear, transferring load primarily to the lap plates adjacent to the main plate cutoffs. The double shear behavior in Region 1 was also evident in Joints 1, 4, 5 and 7.

Figure 19 summarizes the measured hole offsets in Joint 3A at the 1000 kip load level just prior to failure at 1030 kips. Failure occurred by a complete shearing of all fasteners along the shear plane adjacent to the plate discontinuities. The deformation along the failure shear plane was greater than the deformation along the secondary shear plane. This same condition was observed in Joints 2 and 3B. All of these joints had an A_n/A_s ratio of 1.13 (assuming double shear) and exhibited rigid plate behavior. It was apparent that these short stiff joints did not redistribute load as readily to other plate elements as the more flexible joints.

4.3 Comparison of Theoretical Solution to Test Results

The theoretical ultimate strength of the test joints was determined assuming, (a) complete double shear behavior, and (b) double shear behavior in Region 1 with single shear in the interior regions. In Table 4-B, the theoretical predictions are compared with the test results.

Except for Joint 2, the predicted ultimate loads assuming complete double shear were within 10% of the experimental values. The largest variations were in the shorter joints with high A_n/A_s ratios. The strengths of these joints were overestimated.

The tests showed (Fig. 18) that double shear did occur in the first region but that single shear was more evident in the interior regions. The analytical predictions of the joint strengths assuming this type of behavior were comparable to the predictions assuming complete double shear in Joints 1, 4, 5, 6, 7, and 8 (compare columns 4 and 5 in Table 4-B). In Joints 2, 3A, and 3B with relatively high A_n/A_s ratios, a substantial decrease in strength was predicted by assuming single shear in the interior regions. This was in agreement with the experimental results.

As illustrated in Fig. 19, Joint 3A exhibited rigid plate behavior along its length. The measurements indicated that a double shear condition did not exist near the ultimate load. Assuming double shear resulted in an overestimate of the joint strength by about 10%. The theoretical predictions for Joints 2, 3A, and 3B, assuming single shear in the interior regions, were slightly conservative compared to the test results.

Columns 6 and 7 of Table 4-B compare the A_n/A_s ratios of the test joints that correspond to the assumptions of complete and modified double shear. An increase in A_n/A_s ratio occurred by reducing the effective fastener shear area. The A_n/A_s ratio, 0.75, in Joint 1 assuming double shear, corresponds to an effective shear stress of about 22 ksi at the plate working load level. For the observed behavior, i.e., single shear in the interior regions, the effective shear stress is 34 ksi. This is only slightly greater than the recommended value of 30 ksi for bearing type joints suggested in Ref. 4. The predicted strengths for both types of behavior were comparable and slightly less than the test results for Joint 1. This agrees with the observations in the analytical studies that shingle joints would behave satisfactorily if designed for complete double shear using the conservative current specification of 22 ksi.

The effective shear stress in Joints 2, 3A and 3B corresponding to double shear in Region 1 with single shear in the interior regions is 45 ksi. Thus, the geometries of these joints are not likely to occur since excessively high allowable shear stresses would result. It is also unlikely for a joint with only 12 fasteners in line to be designed as a shingle joint.

The analytical studies predicted that varying the region lengths by shifting fasteners into the first region would not greatly influence the ultimate strength. In the study, double shear was assumed in Region 1 with single shear in the interior regions. In certain cases, failure was predicted in the interior fasteners.

In the test program, Joint 5 had 4 fasteners shifted into the first region. Joint 1 had equal region lengths. The predicted ultimate strengths for Joints 1 and 5 are compared in Table 4-B. When single shear was assumed in the interior regions, a slight decrease in strength was predicted by rearranging the fasteners. A predicted interior fastener failure also resulted. Assuming double shear throughout the joint resulted in a slight increase in predicted strength.

The two theoretical predictions bounded the recorded ultimate load of 1142 kips for Joint 5. The prediction assuming single shear in the interior regions was slightly conservative. An interior fastener failure was observed as shown in Fig. 16. The variation in recorded strength between Joints 1 and 5 of about 6% was in reasonable agreement with the trend observed in the analytical studies.

Figure 11 compares the predicted load-deformation curve with the test results for Joint No. 1. The predicted joint deformation was determined by integrating the computed bolt and plate deformations along the joint length. After the first major slip load, the measured slip was added to the computed deformation. The theoretical curve followed the test results up to the predicted ultimate load of 1149 kips. The joint sustained further loading and continued to deform until failure occurred in the end fasteners at 1210 kips. The predicted ultimate strength was within 5% of the experimental value.

Figure 20 compares the predicted distribution of load in the main and lap plates with the measured plate forces at the ultimate load level. Good agreement exists along the total length of the joint.

4.4 Comparison of Joint 7 and 8 with the Modified Bolted Joint

Table 4-B compares the predicted behavior of Joints 7 and 8 with a gage strip from the Modified Bolted Joint.[9] The Modified Bolted Joint had 16 fasteners in line with a distribution of 5, 5 and 6 fasteners per region. With comparable plate areas, Joint 7 had twice the total number of fasteners, and Joint 8 had twice the number in the first region. For direct analytical comparison, the material properties of Joints 7 and 8 were assumed in predicting the ultimate strength of the Modified Bolted Joint.

The effective shear stress in Joint 7 at the plate working load level assuming double shear in Region 1 and single shear in the interior regions was about 14 ksi. This was comparable to the stress commonly used in bridge joints. In the Modified Bolted Joint the stress was analogous to that of a bearing type joint using the allowable stress recommended in Ref. 4.

The ultimate strengths were predicted assuming (a) complete double shear and (b) double shear in Region 1 with single shear in the interior regions. The predictions for the two types of behavior were comparable as shown in Columns 4 and 5 (Table

4-B). For modified double shear behavior an increase in strength of about 12% was predicted by doubling the number of fasteners in the Modified Bolted Joint. The same effect was predicted by doubling the number of fasteners in only the first region. In each case bolt failures were anticipated.

The experimental results were in good agreement with the analytical predictions. As reported in Ref. 3, the predicted strength of the complete Modified Bolted Joint assuming double shear was within 5% of the experiment load of 3550 kips.

It was concluded that the strength of large bridge joints would not be substantially decreased by removing up to half the number of fasteners currently used in design.

4.5 Behavior of Huck-Fastened Joints

Joints 2 and 3B were fastened with Huckbolts. Joints 3A and 3B were identical except for the type of fastener.

The average clamping forces for the various bolt lots are listed in Table 3. The clamping forces developed by the Huck-bolts were estimated by installing several fasteners in a Skidmore-Wilhelm calibrator. The average force per bolt of 45.6 kips was about 17% higher than the specified minimum. The total joint clamping forces listed in Table 4-A for Joints 2 and 3B were estimated from this average value. The clamping forces in the

A325 bolts were determined from the measured elongations of bolts installed in the joints with 1/2 turn-of-nut. These forces were as much as 30% higher than the average Huckbolt value determined in the Skidmore-Wilhelm.

It was observed in Ref. 2 that variations in the stiffness of the connected material can effect a variation in Huckbolt clamping force. Higher clamping forces were found in assemblies having the least amount of compressive deformation. Since a well compacted joint is stiffer than the Skidmore-Wilhelm calibrator, it is believed that the actual clamping force developed in the test joints was 10-15% higher than assumed.

The load-deformation results of Joints 3A and 3B are compared in Fig. 14. The various stages of slip for both joints are shown. As expected, slip occurred at a lower load in the Huck-bolted joint due to the slightly lower clamping forces. The difference in slip loads was about 12%. The slip coefficient assuming two shear planes for Joint 3B was 0.407 when the calibrator was used to estimate the clamping force. Since the joint clamping forces are probably greater, the apparent increase in slip coefficient was expected.

The slip behavior of the Huck-bolted joints was comparable to the behavior observed in the A325 bolted joints. Slightly lower clamping forces were found in the Huck fasteners than in the A325 bolts tightened by the turn-of-nut method. The clamping forces induced in the Huckbolt fasteners however, were in excess of the minimum requirements of A325 bolts.

The shear jig tests results listed in Table 3 showed the double shear strength and ultimate deformation to be nearly identical for the A325 bolts and Huckbolts. Joints 3A and 3B had the same geometry and their predicted and measured ultimate loads were nearly identical for both types of assumed behavior as shown in Table 4-B. Figure 14 compares the experimental load-deformation results for these two joints. As expected, the behavior of Joint 3B was nearly the same as Joint 3A. The Huck-bolted joint yielded a slightly higher ultimate strength than the A325 bolted joint. The correlation between the predicted ultimate strengths and the measured strengths were comparable for the Huck and A325 bolted joints.

5. DESIGN OF SHINGLE JOINTS

5.1 Approximate Methods of Analysis

Shingle joints like other types of connections are statically indeterminant, thus, the distribution of forces depends upon the relative deformations of the component members and fasteners. The condition is further complicated in shingle joints by the unsymmetric positioning of main plate terminations. Analytical elastic solutions that predict the distribution of load in the main and splice plates of shingle joints have been developed.[9] The solution has been extended into the plastic range so as to predict the ultimate strength of the connection.[3] These theoretical analyses however, are too cumbersome and impractical for ordinary design practice. Simplifying assumptions must be made that reduce the solution for design to one based primarily on equilibrium.

There are several existing methods for estimating the distribution of force in the main and lap plates of a shingle splice. Two of the most popular methods are:[12]

1. Forces in splice plates are inversely proportional to their distances from the member being spliced.
2. Forces in each member at a section through a splice are proportional to their areas.

In Method 1, it is assumed at each discontinuity that the amount of force distributed to the lap plates is proportional to the area of the member being terminated. The forces in the continuous main members are assumed to remain unchanged. This is illustrated schematically in Fig. 21-a. The transfer of load is made in the region directly preceding the point of termination and it is assumed that the original load is restored to the spliced member in the region following the termination.

In Method 2, (see Fig. 21-b) the total applied load is assumed to be distributed to all continuous members at the position of a main plate termination in proportion to their areas. No direct assumption is made regarding the amount of load transferred to the splice plates in a particular region as in Method 1. If the lap plates are of equal area, Method 2 predicts that the shear transfer is equal along the top and bottom shear planes in the first region regardless of their positions with respect to the member being terminated.

Previous shingle joint tests have shown that at each plate discontinuity, there was a sudden pick-up of load in the adjacent plate elements.[6,9] Another approximate method of analysis was developed on the basis of this earlier observation and these test results. This method, referred to as Method 3, and illustrated in Fig. 21-c, assumes that the total load is distributed to all members at a section through the joint in proportion to their areas, first considering the terminated members as being continuous. The load assumed to be carried by a terminating member is then distributed to the two adjacent plates in proportion to their areas. Hence a two stage distribution is used.

5.2 Comparison of Design Methods with Test Results

The partition of load in the test joints was determined from the measured plate strains at different cross sections along the joint lengths.

Figures 22, 23 and 24 compare the measured plate forces in Joints 1, 4 and 8 with the various design methods. Similar distributions were found in the other test joints. The comparisons were made at the working load levels as determined by the main plate net areas.

The top portion of each figure compares the design curves with the measured forces in the combined top lap plates. The central portion of each figure makes a similar comparison for the main plate component, and the lower portions compare the results for the lower lap plates.

In Fig. 25, the various methods for design are compared with the test results reported in Ref. 9 for the Modified Bolted Joint. The comparisons are made for the top lap plate, main plate and bottom lap plate components at the 2080 kip working load level.

The geometry of the Modified Bolted Joint differed from the geometries of the recent test joints in that it had two splice plates along the bottom shear plane. Only a single bottom splice plate was used with the test joints reported herein.

It is apparent from the comparisons of load in the main plates assumed by Method 1 with the test results, that Method 1 substantially underestimated the total transfer of load in the frist region. The measured load transferred to the splice plates always exceeded the proportion of main plate area initially being terminated. In Joints 1, 4 and 8 (Figs. 22, 23 and 24), 50% of the applied load was transferred to the lap plates in the first region although only 33% of the main plate was terminated. In the Modified Bolted Joint, over 50% of the applied load was initially distributed.

Loads substantially greater than estimated by Method 1 were measured in the bottom lap plates of all joints. The test results indicated that the forces in the top and bottom lap plates were nearly equal in the first region. In the Modified Bolted Joint more load was actually measured in the combined bottom lap plates than in the top lap plate as shown in Fig. 25.

The critical shear plane determined by Method 1 is always the plane adjacent to the main plate terminations. Since less fasteners are required along the bottom shear plane, the bottom lap plates are often shortened in the first region.

However, a condition close to double shear exists in the first region because of the large amount of force transferred to the bottom lap plates. This was illustrated in Fig. 18 by the sawed section of Joint 8 after failure. Equal deformation was observed in the end fasteners along both shear planes. Thus, shortening the bottom lap plates and eliminating fasteners from the bottom shear plane is a wasteful practice which does not fully utilize the fastener.

The greatest variation between the load partition determined by Method 1 and the test results occurred in the Modified Bolted Joint (Fig. 25). It was apparent that the assumptions in Method 1 used to determine the distribution of force to the lap plates were not very satisfactory.

The distributions of load in the main plates of the joints determined by Method 2 were in good overall agreement with the measured forces. In joints 1, 4 and 8, it was estimated that 50% of the load would be distributed to the lap plates in the first region. In the Modified Bolted Joint (Fig. 25), because of the greater proportion of splice material, it was estimated that 59% of the load would be distributed. Both assumed distributions were comparable to the test results.

Slight variation between the distributions determined by Method 2 and the test results occurred in the top and bottom lap plates. The forces in plates adjacent to a plate termination were slightly underestimated in all test joints. The greatest deviations were observed in the top lap plates adjacent to the first plate terminations and in the bottom lap plates adjacent to the final plate terminations. Increases in plate loads occurred at those points. Reasonable agreement was apparent between the distribution determined by Method 2 and the test results for design purposes.

The distributions of force determined by Method 3 provided the best correlation with the test results. This is shown for Joints 1, 4 and 8 in Figs. 22, 23 and 24 and also for the more complex Modified Bolted Joint in Fig. 25. The method provided a reasonable estimate of the force distributions in all joint components.

6. SUMMARY AND CONCLUSIONS

These conclusions are based on the results of a series of analytical studies and a complimentary experimental program that examined the behavior and ultimate strength of shingle joints. The theoretical solution for shingle joints reported in Ref. 3 was used to make the analytical studies. In the test program, nine shingle joints of A572 steel were tested. Two joints were fastened with 7/8 in. Huckbolts. The remaining joints were fastened with 7/8 in. A325 bolts.

(1) The analytical studies showed that A325 bolts were capable of satisfactory behavior at allowable shear stresses up to 40% higher than used in current practice.

(2) The predicted strengths of shingle joints proportioned with the fasteners in double shear at a 22 ksi allowable stress were not substantially altered when the joints were assumed to have single shear in the interior regions. The resulting effective allowable stress was 34 ksi.

(3) For shingle joints of a given length, the predicted strength was not greatly influenced by shifting fasteners into other regions.

(4) At high A_n/A_s ratios, one-region butt joints were predicted to be more efficient than shingle joints with two and three regions due to the complete double-shear behavior and constant joint stiffness in the butt joints. At lower A_n/A_s ratios, the number of regions had no effect upon the predicted strength.

(5) The test joints normally exhibited two separate load levels or stages at which major slip occurred. First slip occurred along the shear plane adjacent to the main plate terminations.

(6) The test results confirmed the indications of previous studies that the total rigid body movement in shingle joints tends to be less than the hole clearance. Since shingle joints are most often used where reversal of stress is unlikely because of large dead loads, it appears reasonable to assume that shingle joints are not slip-critical.

(7) The slip coefficients computed on the assumption of double shear in the first region and single shear in the interior regions were in better agreement with other test data for clean mill scale surfaces.

(8) Two distinct types of behavior in the non-linear range were encountered. Joints with high A_n/A_s ratios experienced relatively little non-linear deformation and resulted in multiple-bolt shear failures. In joints with lower A_n/A_s ratios, large deformations were measured after gross section yielding. End-bolt shear failures or plate failures were observed.

(9) The tests showed that double shear did occur in the first region but that single shear was more evident in the interior regions. In the test joints which had an effective allowable shear stress of 34 ksi or less corresponding to this type of behavior, the predicted strengths assuming complete double shear were within 5% of the experimental values.

(10) It was verified by the test results that the theoretical solution reported in Ref. 3 could accurately predict (a) the ultimate strength, (b) the load-deformation behavior, and (c) the distribution of load in the main and combined lap plates of shingle joints.

(11) By comparing the predicted strength and comparable test results of Joint 7 with the predicted

strength of a single line of fasteners from the Modified Bolted Joint reported in Ref. 9, it was concluded that the strength of large friction-type joints would not be substantially decreased by removing up to half the number of fasteners.

(12) The slip characteristics and strength of the joints fastened with Huckbolts were comparable to the behavior of joints fastened with A325 bolts.

(13) The load distributed to the splice plates at the position of a main plate termination was more proportional to the area of the splice plates at that point than to the area of the main plate being terminated. A sudden pick-up in load was measured in the plates directly adjacent to a plate discontinuity.

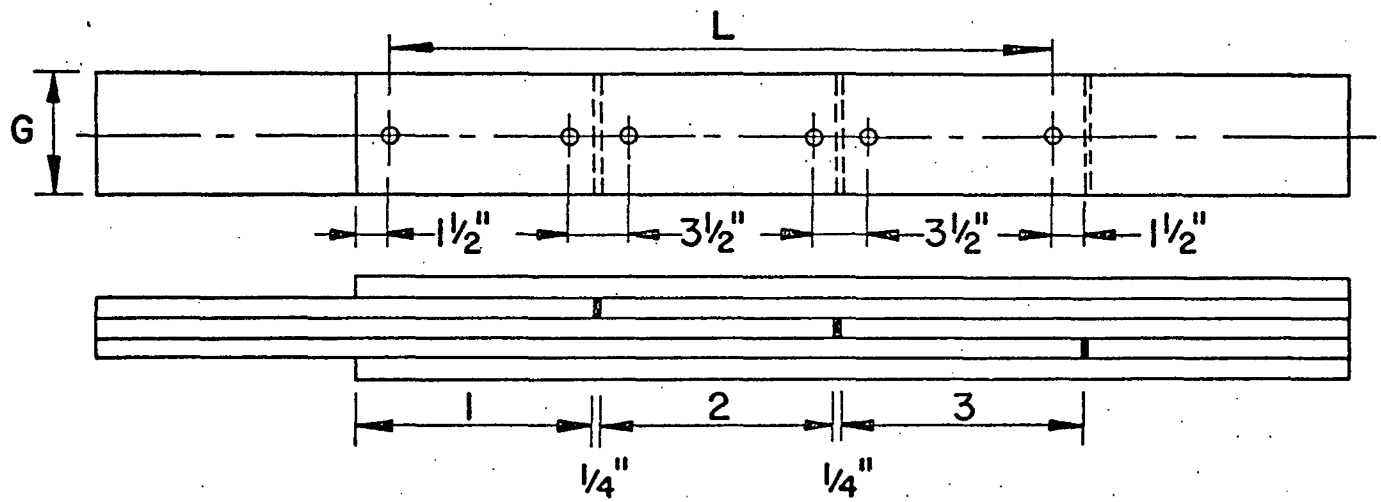
(14) The distribution of load determined by Method 1 can be misleading for design. Since the method substantially underestimates the load carried by the splice plates furthest from the plate discontinuities, excessive numbers of fasteners may result. The practice of shortening the bottom lap plates in the first region was shown to be unfavorable since it did not allow an effective utilization of the fasteners.

(15) A reasonable approximation of the load partition in shingle joints can be made by assuming the force in all continuous members at the position of a main plate termination to be proportional to their areas.

(16) A more exact approximation can be made using the two-stage distribution, described as Method 3, taking into account the sudden pick-up of load in plates directly adjacent to a plate termination. The method accurately predicts a more effective use of fasteners in the interior regions. Thus, less fasteners are required than in the other methods.

7. TABLES

TABLE I-A DESCRIPTION OF TEST JOINTS

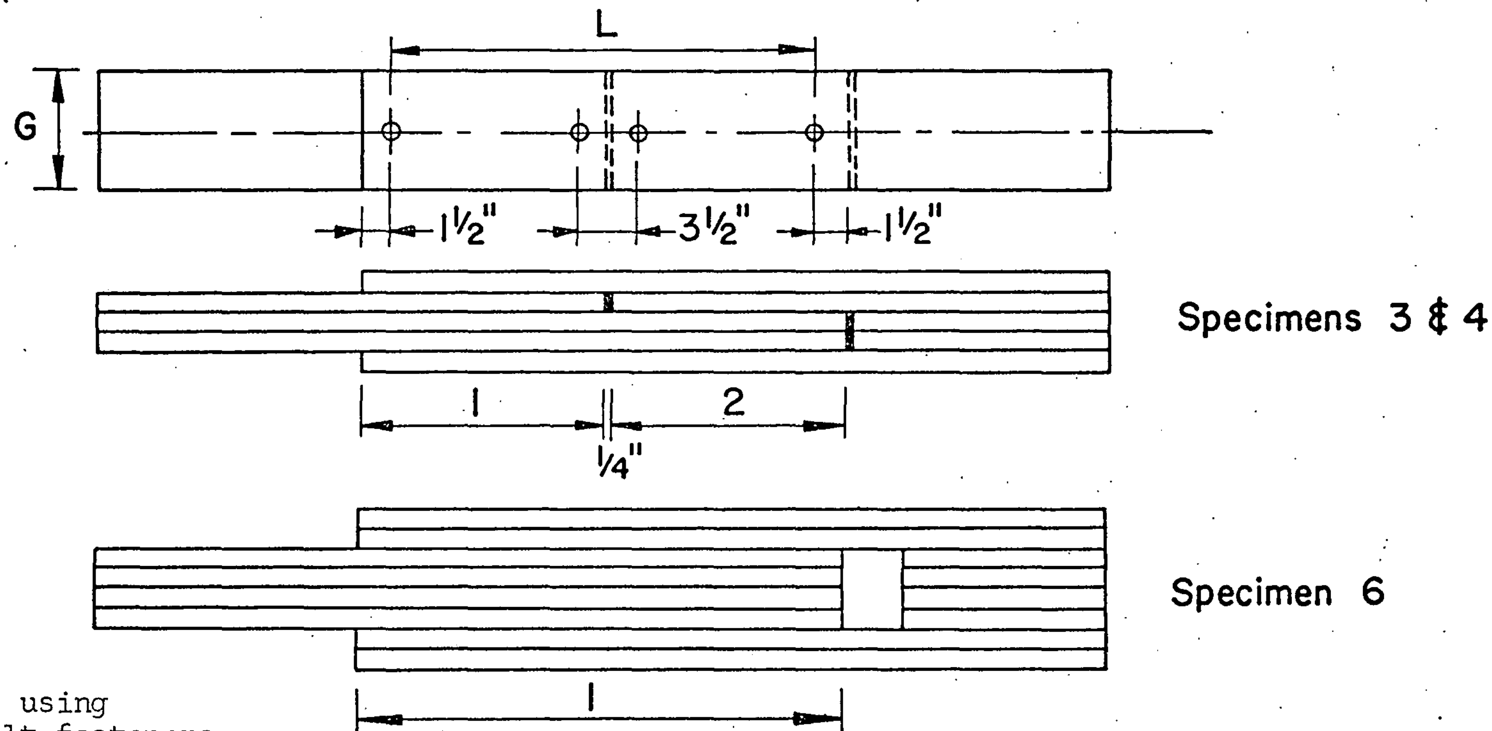


Specimens 1, 2, 5, 7, & 8

* Joints using
Huckbolt fasteners

TEST JOINT	NUMBER OF FASTENERS	FASTENERS PER REGION			LENGTH L (INCHES)	REGION LENGTHS (INCHES)			GAGE G (INCHES)
		1	2	3		1	2	3	
1	18	6	6	6	52.0	18.125	18.250	18.125	6.375
*2	12	6	3	3	34.0	18.125	9.250	9.125	6.375
5	18	10	4	4	52.0	30.125	12.250	12.125	6.375
7	32	10	10	12	94.0	30.125	30.250	36.125	5.000
8	21	10	5	6	61.0	30.125	15.250	18.125	5.000

TABLE I-B DESCRIPTION OF TEST JOINTS



* Joints using Huckbolt fasteners

TEST JOINT	NUMBER OF FASTENERS	FASTENERS PER REGION		LENGTH L (INCHES)	REGION LENGTHS (INCHES)		GAGE G (INCHES)
		1	2		1	2	
3-A *3-B	12	6	6	33.5	18.125	18.125	6.375
4	12	6	6	33.5	18.125	18.125	4.000
6	12	12	-	33.0	36.000	-	5.000

TABLE 2: TEST JOINT AREAS

Test Joint	Region	Main Plates		Lap Plates	
		Gross Area Square Inches	Net Area Square Inches	Gross Area Square Inches	Net Area Square Inches
(1)	(2)	(3)	(4)	(5)	(6)
1,2,5	1	19.125	16.312	12.750	10.875
	2	12.750	10.875	19.125	16.312
	3	6.375	5.438	25.500	21.750
3A,3B	1	19.125	16.312	12.750	10.875
	2	12.750	10.875	19.125	16.312
4	1	12.00	9.188	8.00	6.125
	2	8.00	6.125	12.00	9.188
6	1	20.000	16.250	20.00	16.250
7,8	1	15.000	12.188	10.00	8.125
	2	10.00	8.125	15.00	12.188
	3	5.00	4.063	20.00	16.250

TABLE 3: FASTENER MATERIAL PROPERTIES

Bolt Lot (1)	Used in Joint (2)	Ultimate Double-Shear Kips (3)	Shear Jig Ultimate Shear Stress si (4)	Ultimate Defor-mation Inches (5)	Direct Tension Tensile Strength si (6)	Torque Tension Tensile Strength si (7)	Average Clamping Force Per Bolt Kips (8)
XA	1,3A,4,5	106.0	88.1	0.176	146.2	130.0	58.9
SA	6	107.8	89.5	0.181	144.0	116.9	53.9
G**	M.B.J.	91.5	76.0	0.148	127.7	111.5	51.0
G	7,8	94.1	78.2	0.205	--	99.6	45.5
Huck	2,3B	110.0	91.5	0.166	--	--	45.6*

*Determined from installation in Skidmore-Whilhelm.

**Normal thread engagement, 4½ in. grip.

TABLE 4-A: TEST RESULTS - SLIP BEHAVIOR

Test Joint (1)	Number of Fasteners (2)	Clamping Force Kips (3)	1st Slip Load Kips (4)	2nd Slip Load Kips (5)	K_s Double Shear (6)	K_s Effective (7)
1	18	1060	504	590	0.238	0.356
2*	12	547**	360	470	0.329	0.438
3A	12	703	508	600	0.361	0.482
3B	12	547**	446	504	0.407	0.544
4	12	713	340	--	0.239	0.318
5	12	1055	560	1040	0.265	0.354
6	12	646	366	--	0.283	0.283
7	32	1455	416	--	0.147	0.218
8	21	955	438	574	0.230	0.311

*Joints fastened with Huckbolt Fasteners.

**Determined from mean clamping force of individual fasteners in Skidmore-Whilhelm. Actual clamping forces may be 10-15% higher.

TABLE 4-B: TEST RESULTS - ULTIMATE STRENGTH

Test Joint	Failure Mode	Recorded Ultimate Kips	Predicted Ultimate		A_n/A_s		Effective Shear Stresses	
			Double Shear Kips	Modified Shear Kips	Double Shear	Modified Shear	Double Shear ksi	Modified Shear ksi
(1)	(2)	(3)	(4)	(5)	(6)	(7)	(8)	(9)
1	Bolts	1210	1150	1124	.75	1.13	22.5	34
2	Bolts	982	1129	943	1.13	1.52	34	45
3A	Bolts	1030	1128	904	1.13	1.52	34	45
3B	Bolts	1044	1133	918	1.13	1.52	34	45
4	Plates	754	726	634	.64	.85	19.5	25
5	Bolts	1142	1179	1105	.75	.97	22.5	29
6	Bolts	1150	1128	1128	1.13	1.13	34	34
7	Bolts Plates	962	937	936	.32	.48	10	14
8	Bolts	932	937	936	.48	.65	14	20
M.B.J.	Bolts	888*	841	832	.64	.98	19	29

*Taken as 25% of the test load (1 gage strip).

8. FIGURES

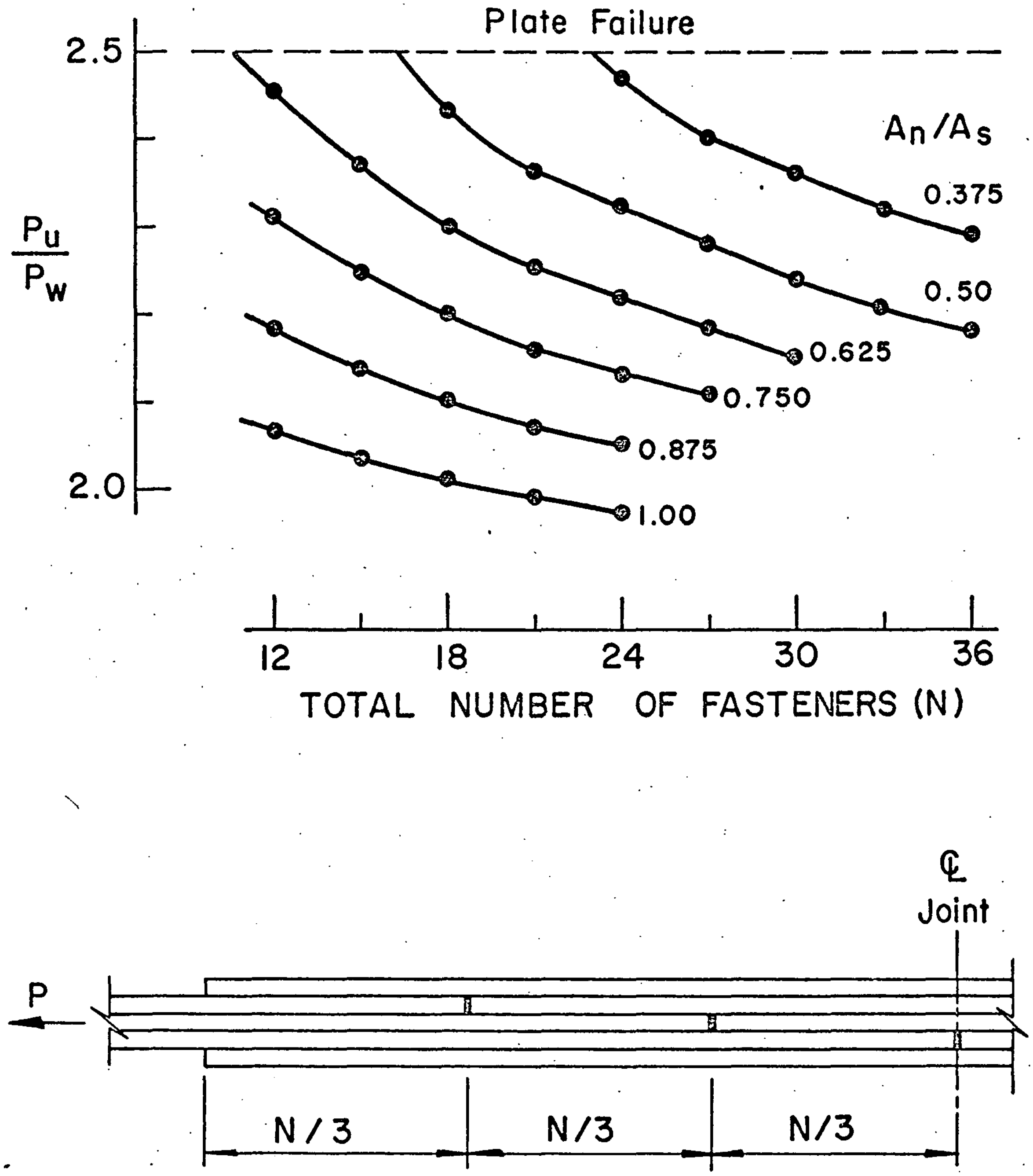


Fig. 1 Effect of Variation in A_n/A_s Ratio and Joint Length Assuming Double Shear

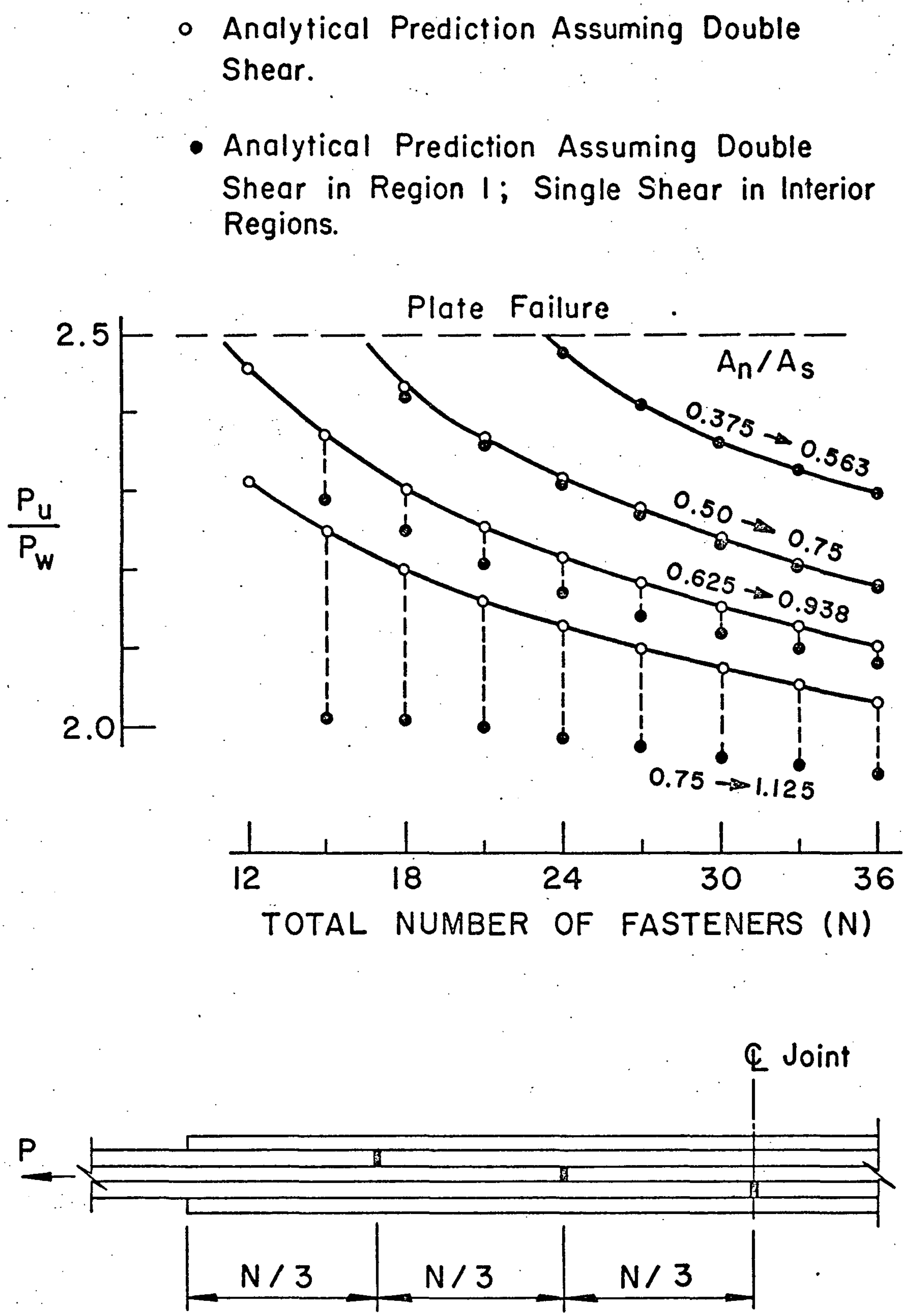


Fig. 2 Effect of Assuming Single Shear in Interior Regions

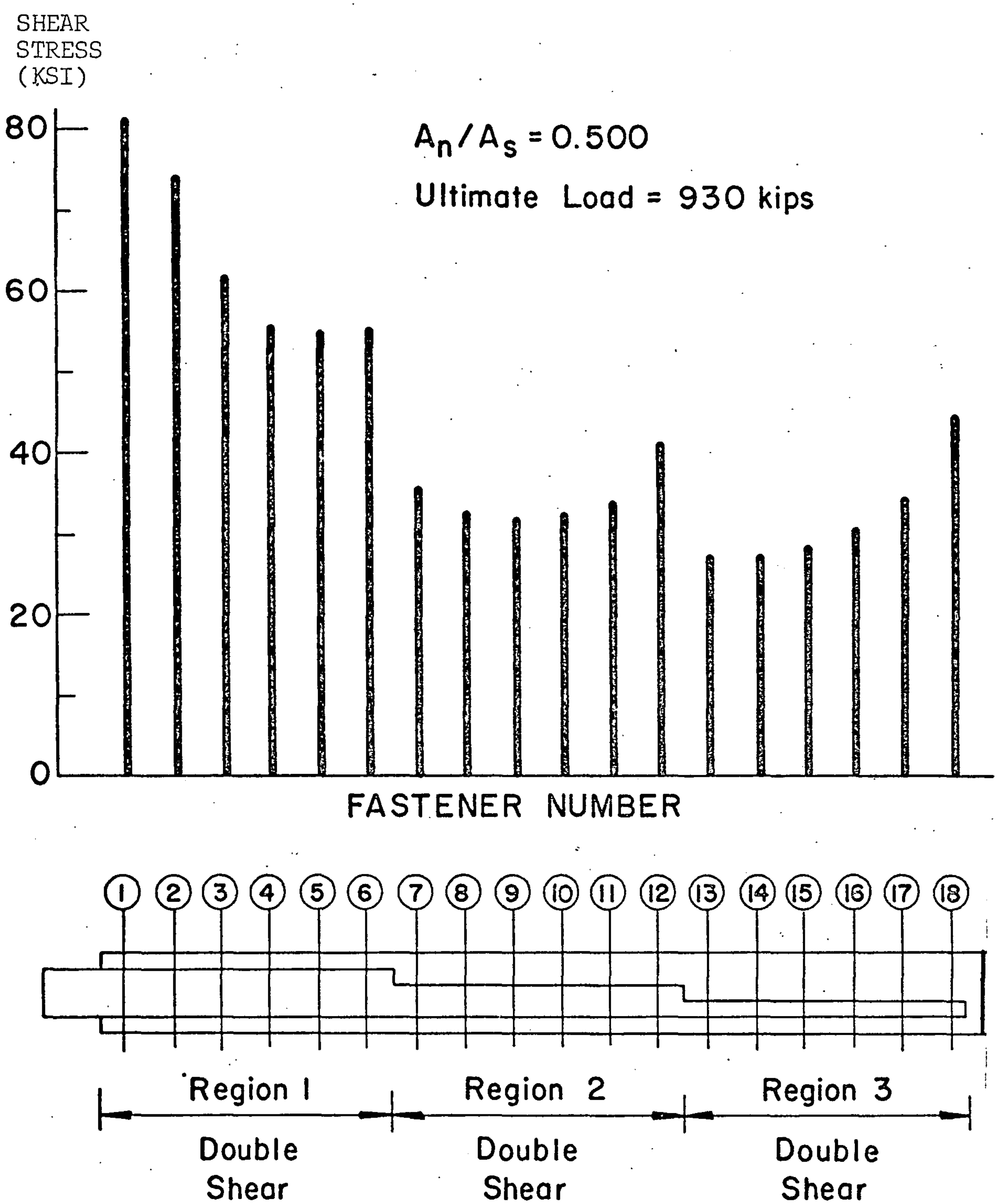


Fig. 3 Fastener Shear Distribution Assuming Double Shear

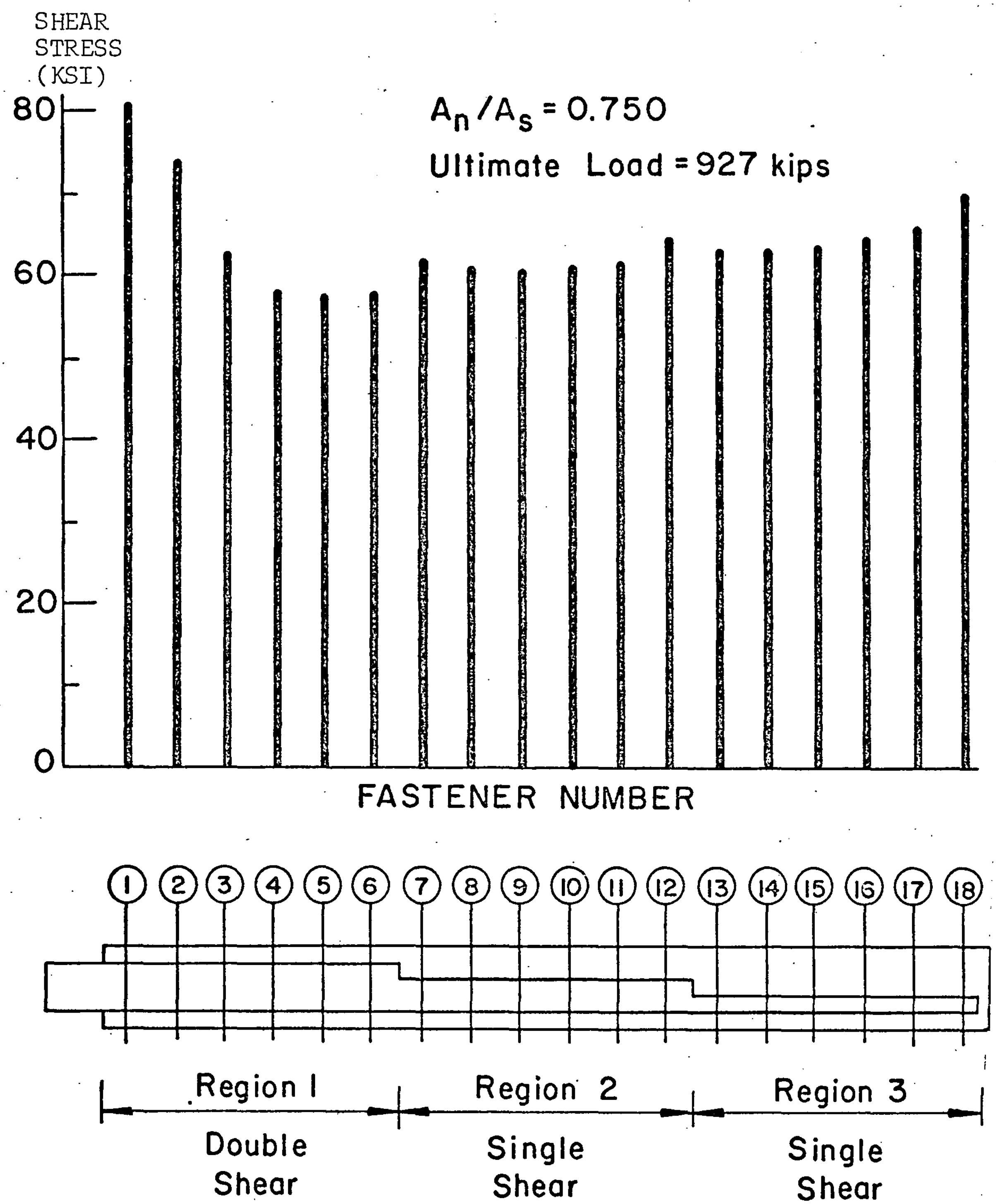


Fig. 4 Fastener Shear Distribution Assuming Single Shear in Interior Regions

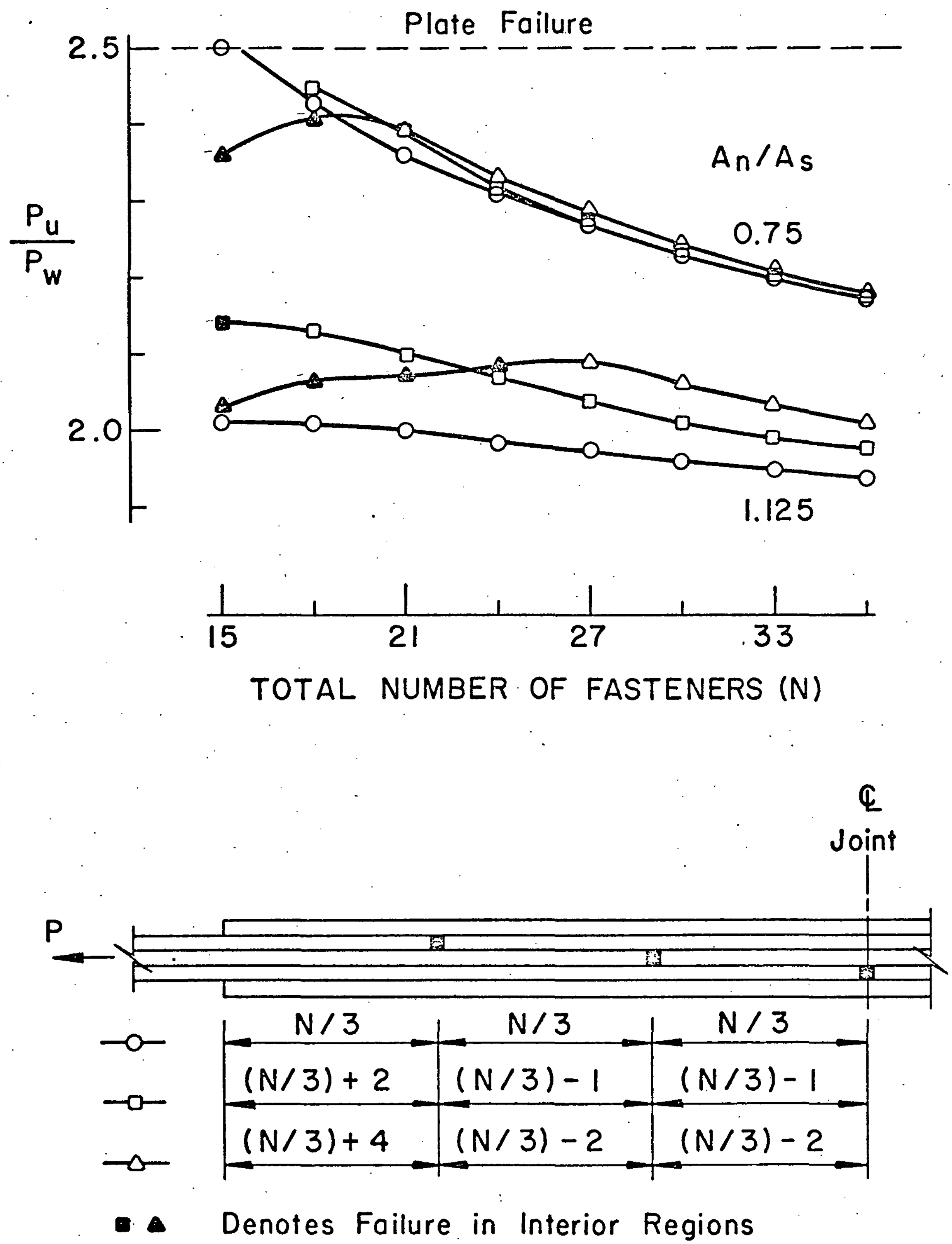


Fig. 5 Effect of Rearranging Fasteners

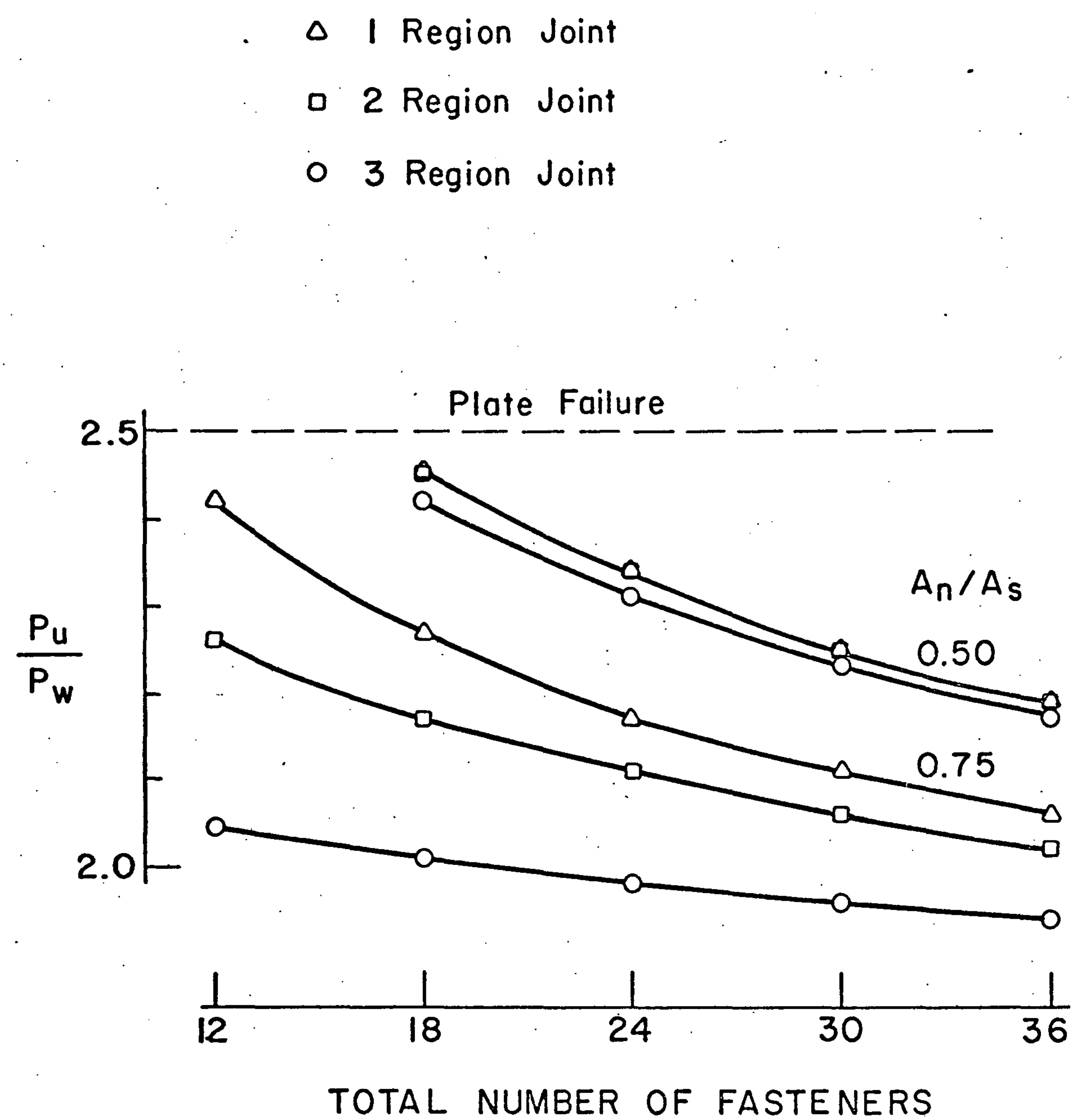
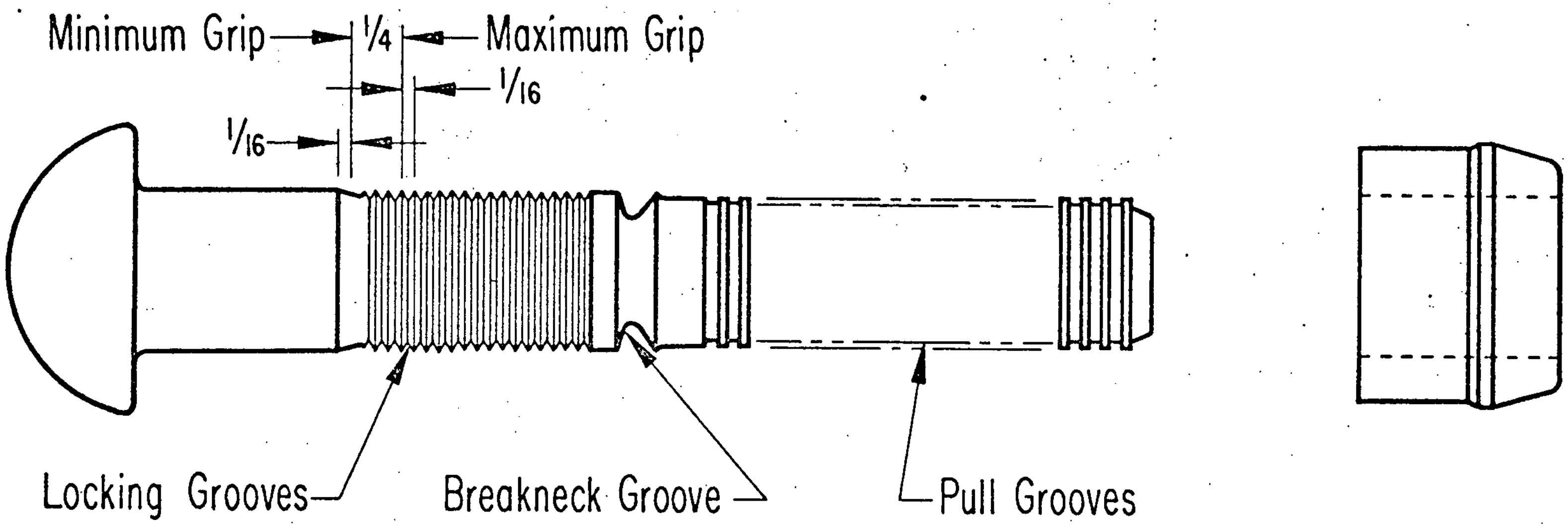


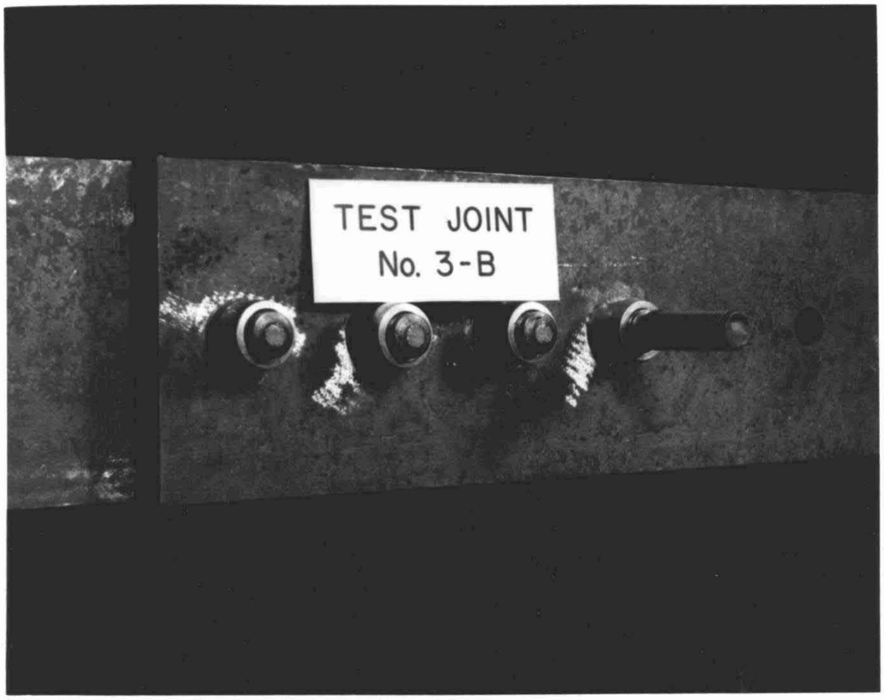
Fig. 6 Effect of Number of Regions



High Tensile Huckbolt Pin

Locking Collar

Fig. 7 Components of High Tensile Huckbolt Fastener



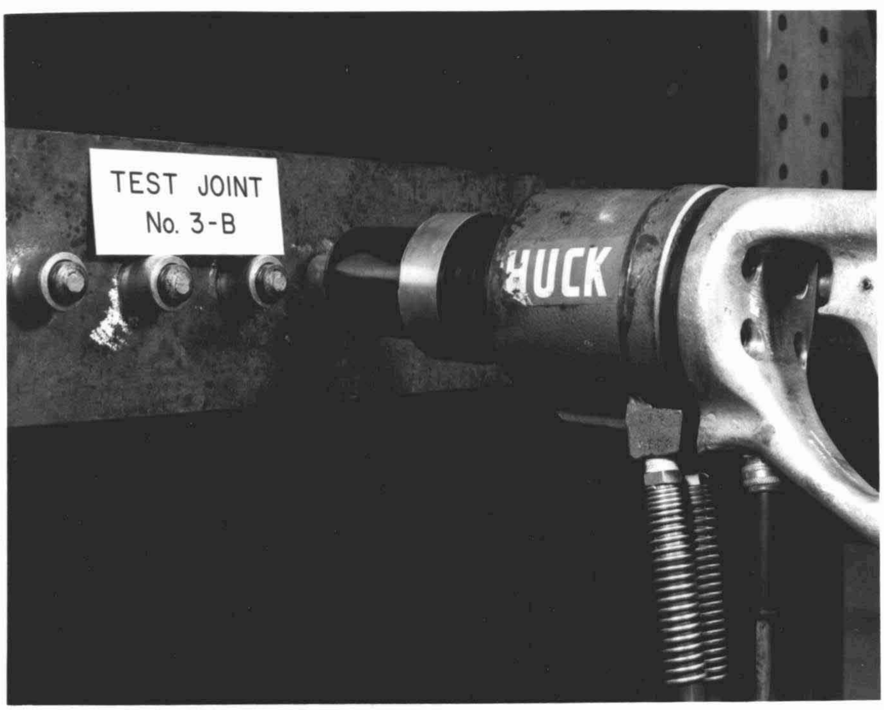
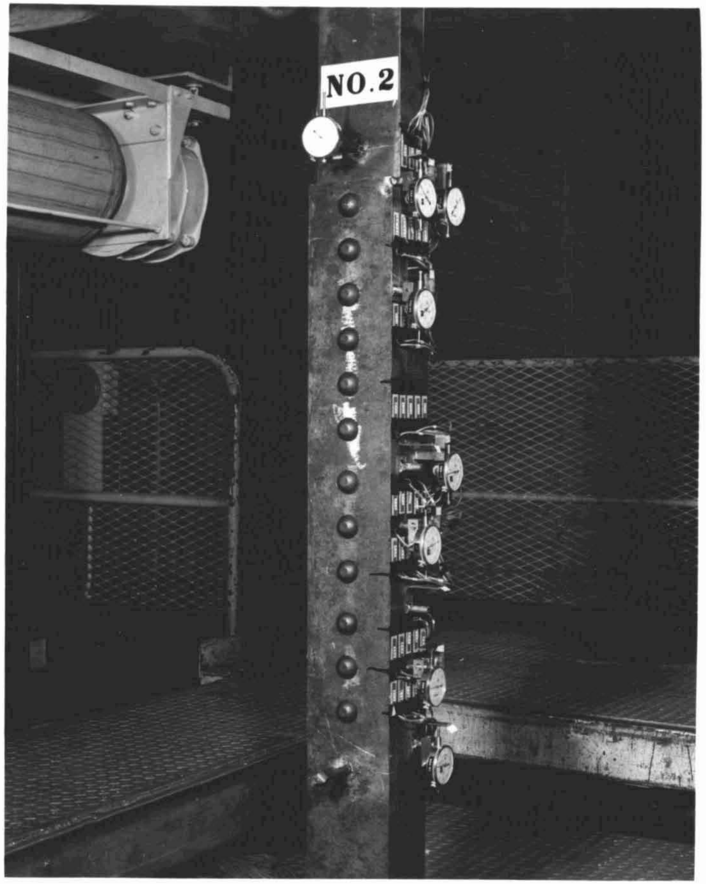


Fig. 8 Installation of Huck Fasteners



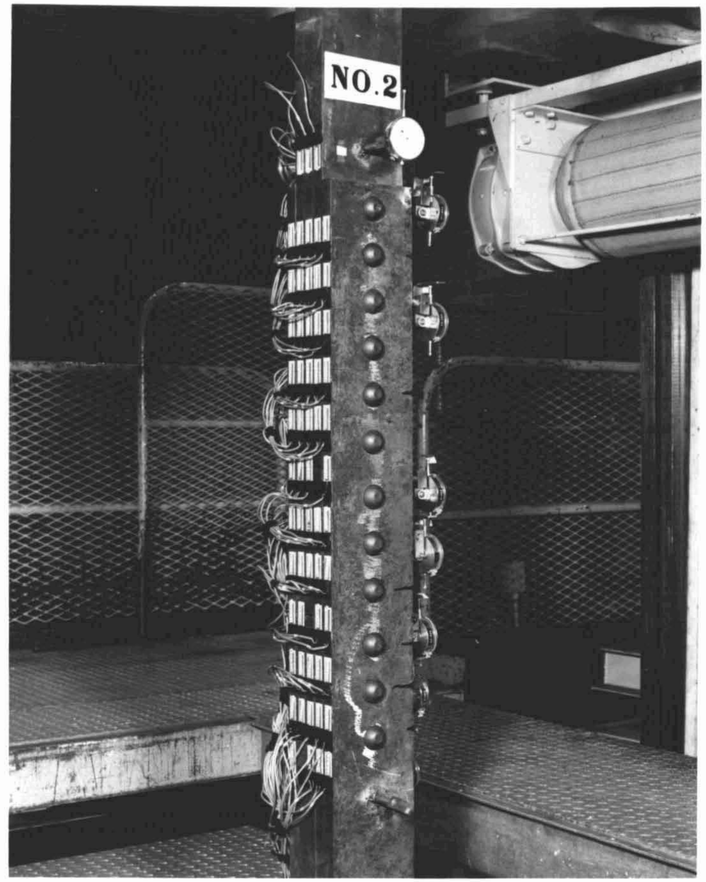


Fig. 9 Typical Instrumentation Set-Up

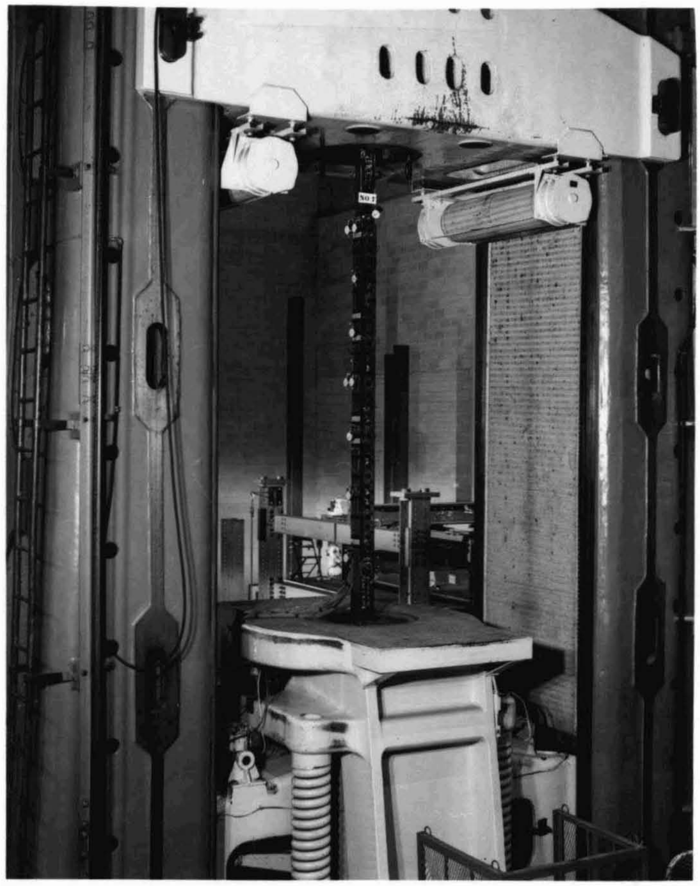


Fig. 10 Test Joint Installed in 5,000,000 lb.
Testing Machine

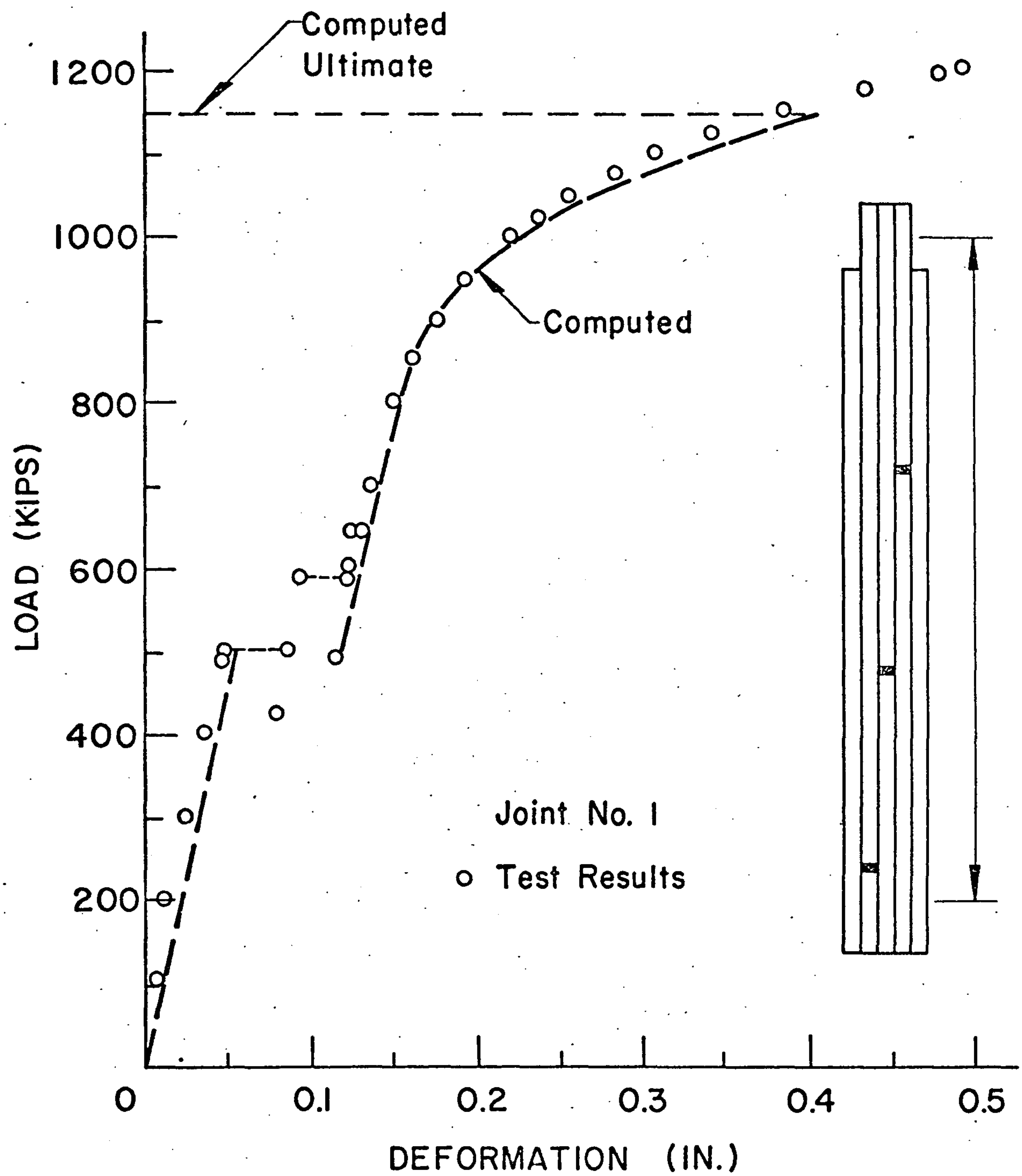


Fig. 11 Computed and Experimental Load-Deformation Behavior

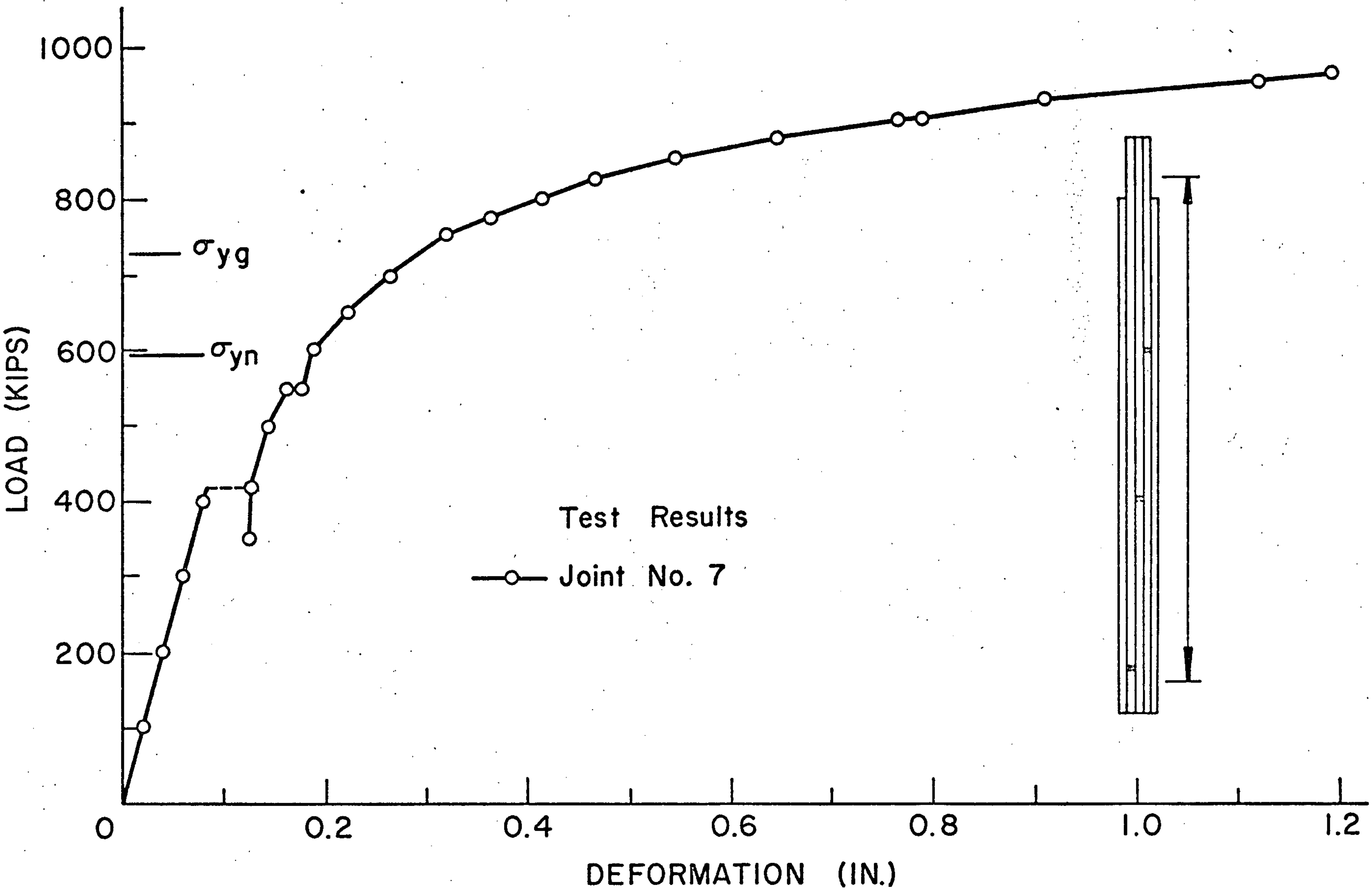


Fig. 12 Load-Deformation Behavior Typical of Joints with Low A_n/A_s Ratios

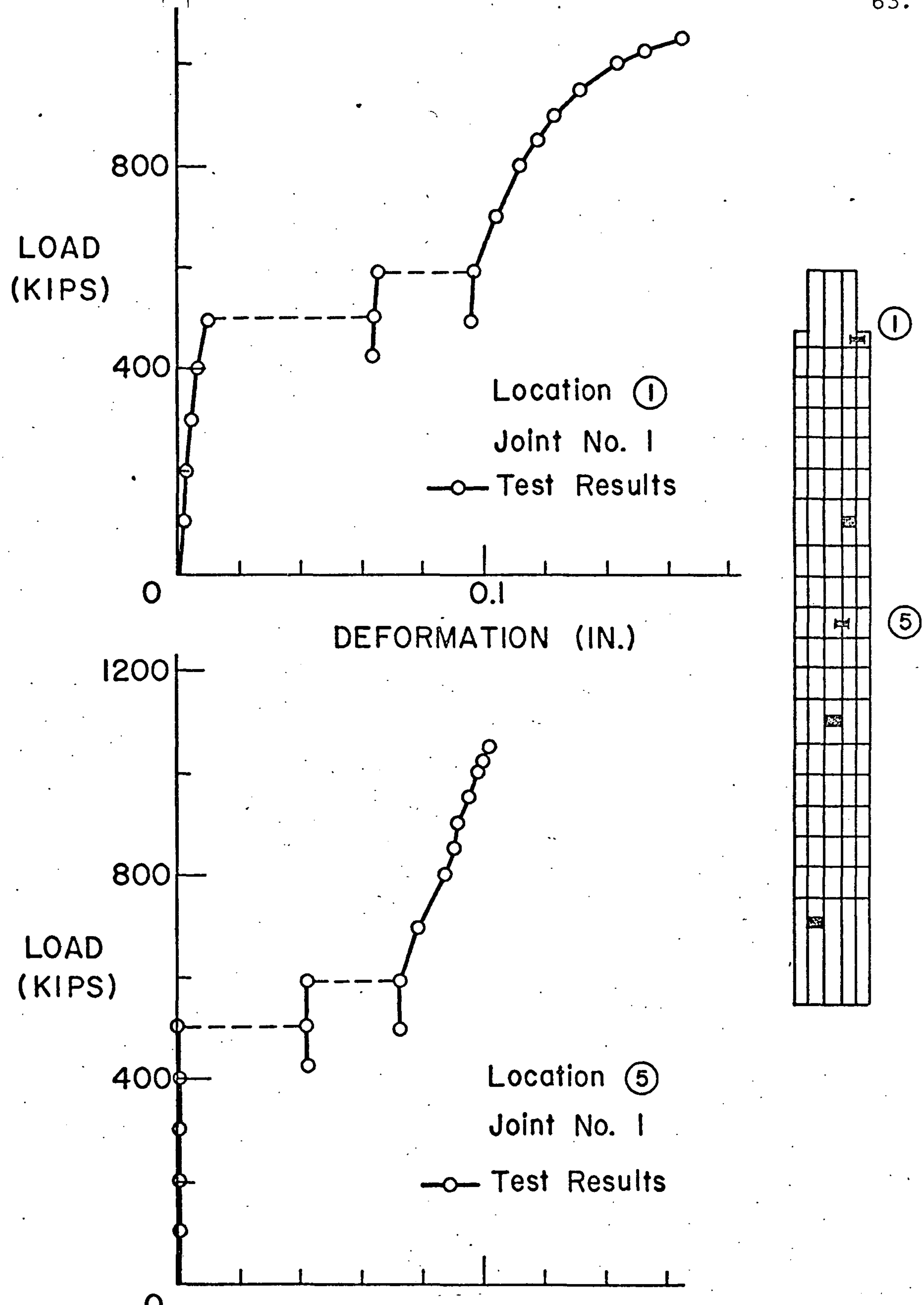


Fig. 13 Local Load-Deformation Behavior

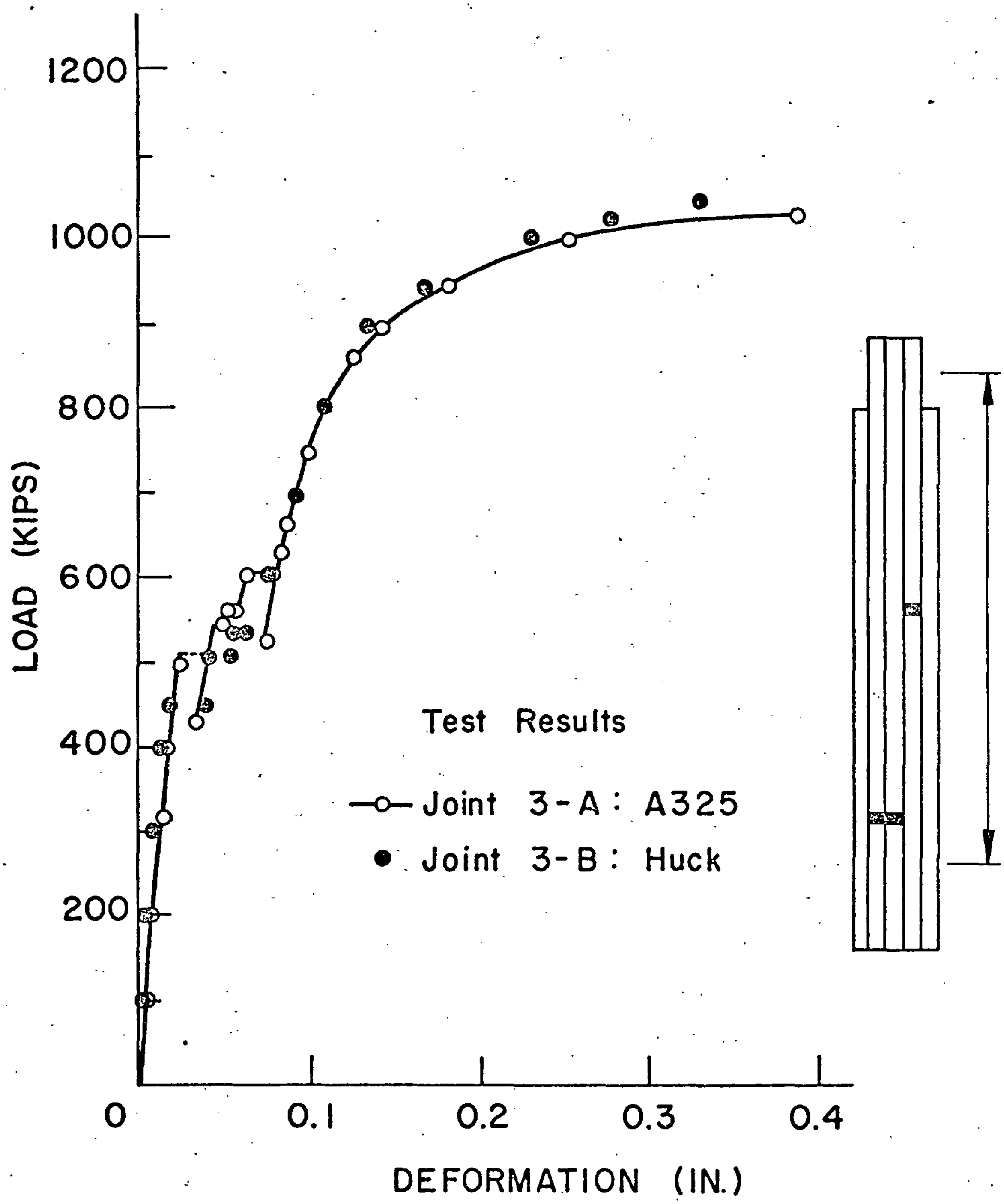


Fig. 14 Comparison of the Load-Deformation Behavior in A325 and Huck-Bolted Joints

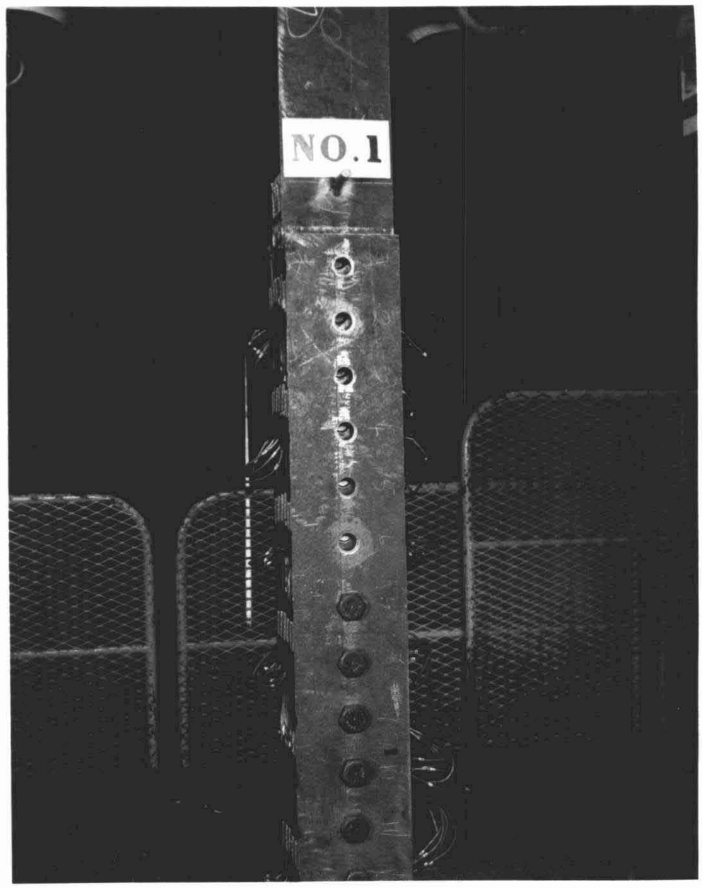


Fig. 15 Shear Failure in Region 1 of Joint 1

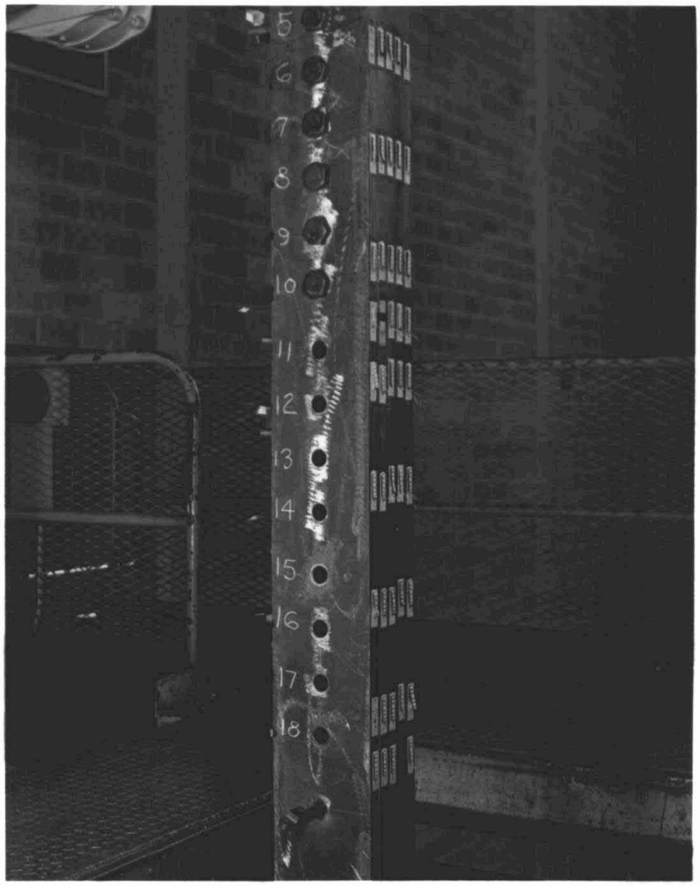


Fig. 16 Shear Failure in Regions 2 and 3 of Joint 5

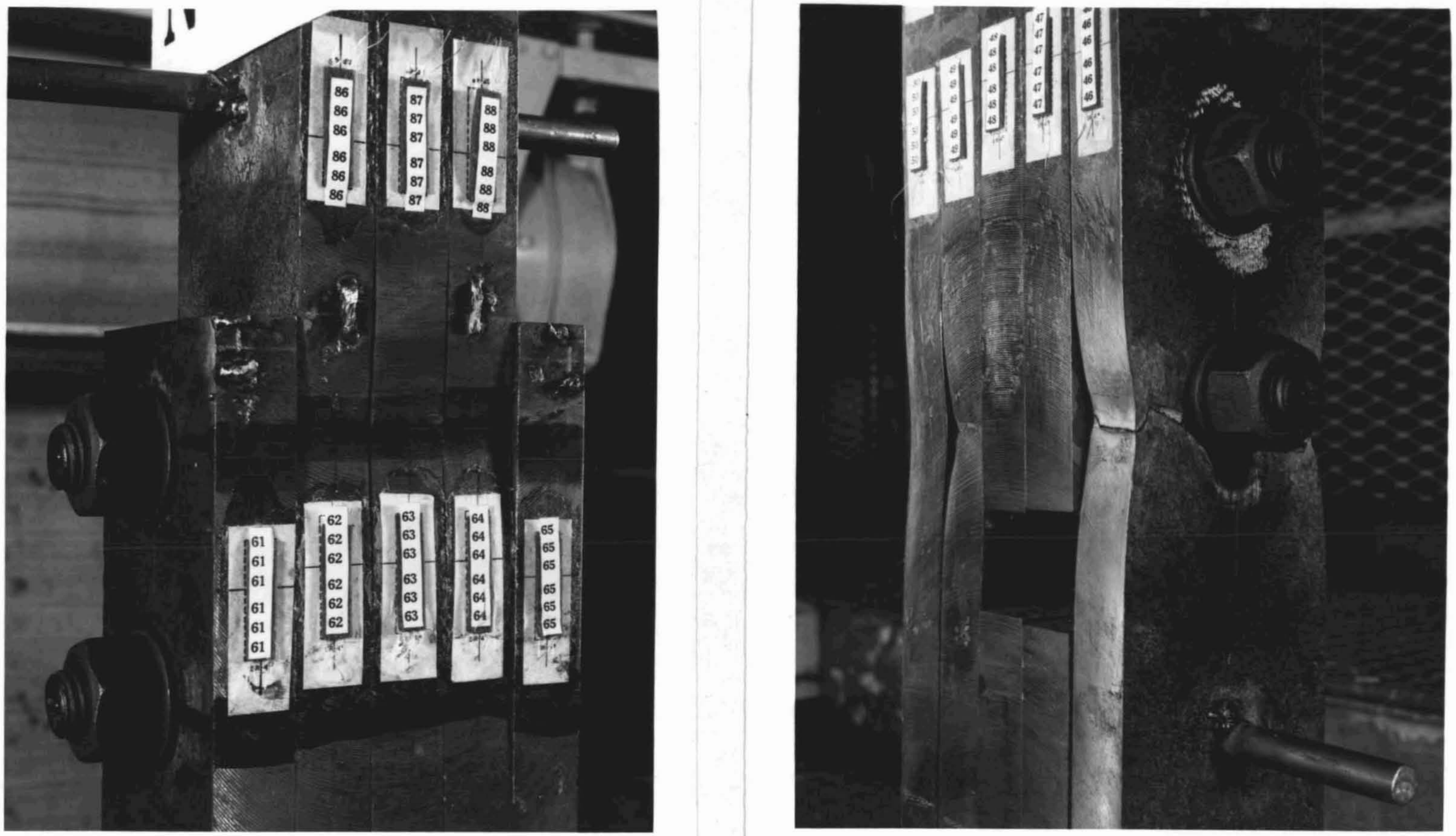
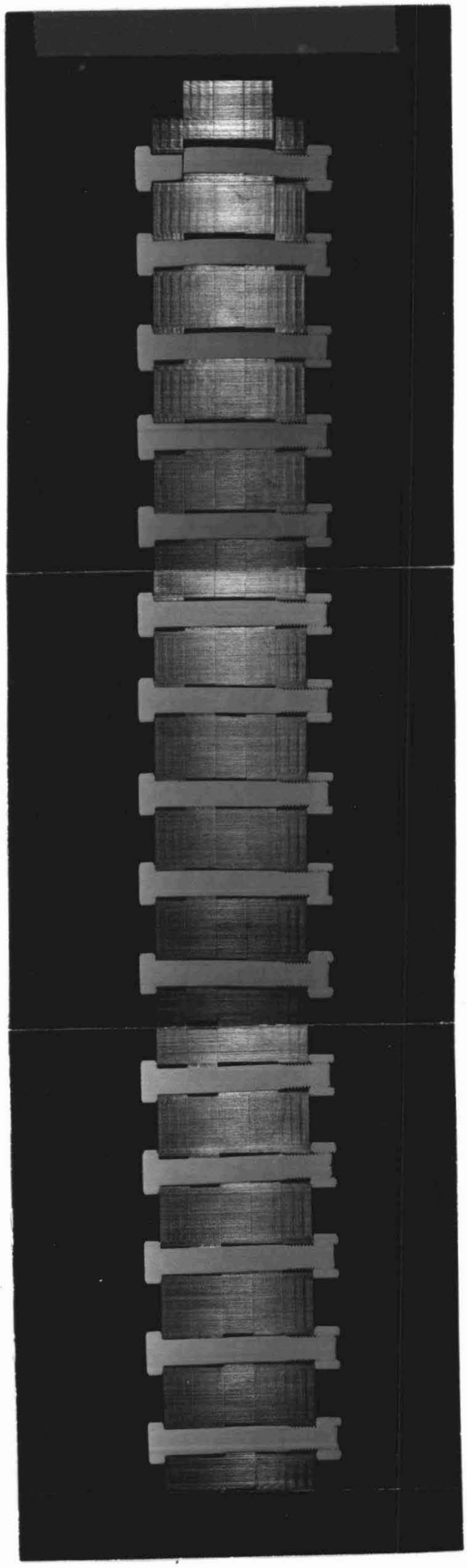


Fig. 17 Plate Failure and Deformation in Joint 4



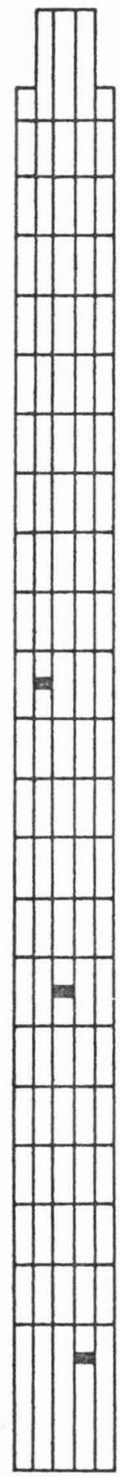


Fig. 18 Sawed Section of Joint 8 After Failure

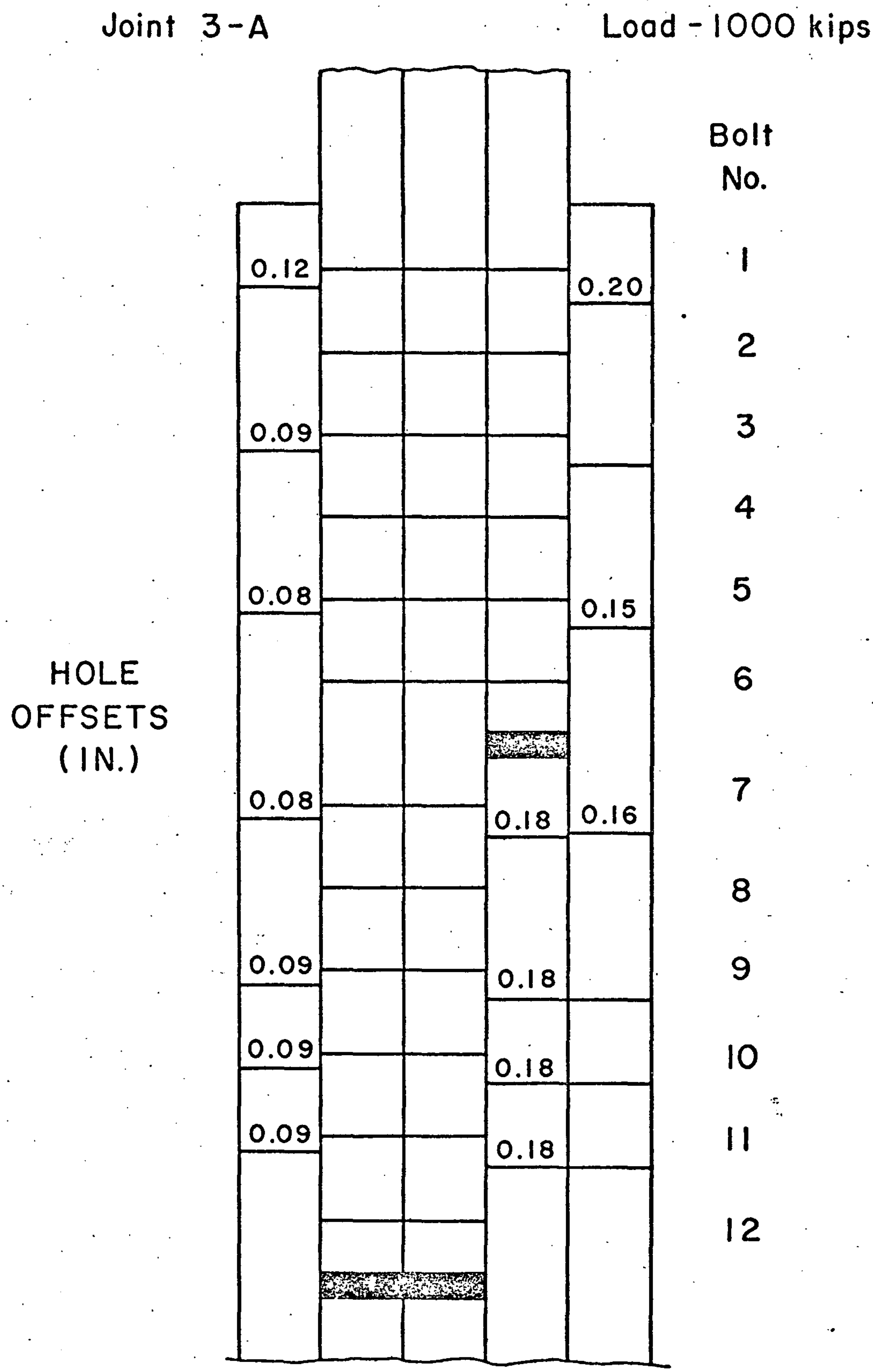


Fig. 19 Measured Hole Offsets on Joint 3A at the 1000 kip Load Level

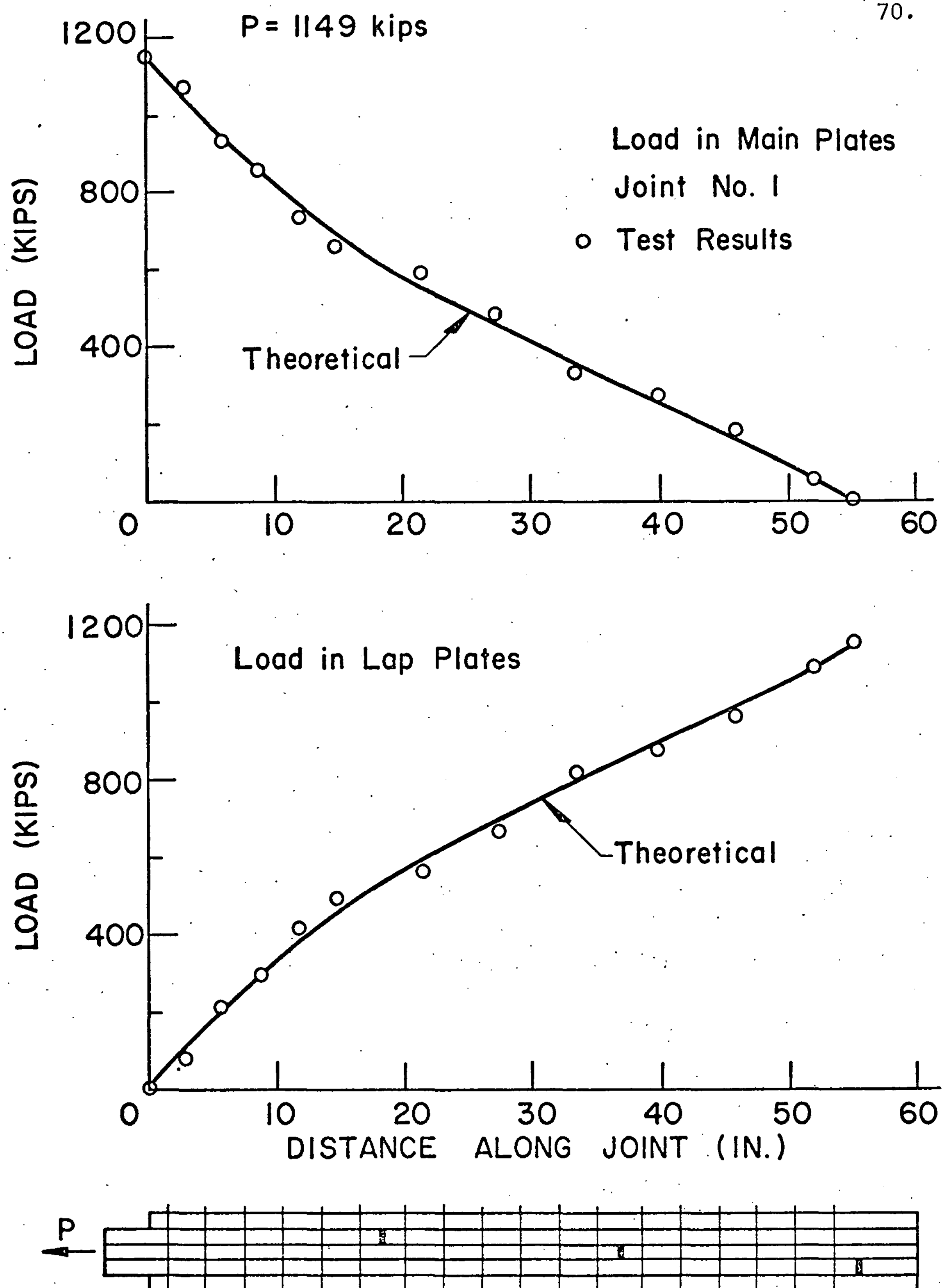


Fig. 20 Load Partition in Joint 1 at the 1149 kip Predicted Ultimate Load

All Plates are of Equal Area

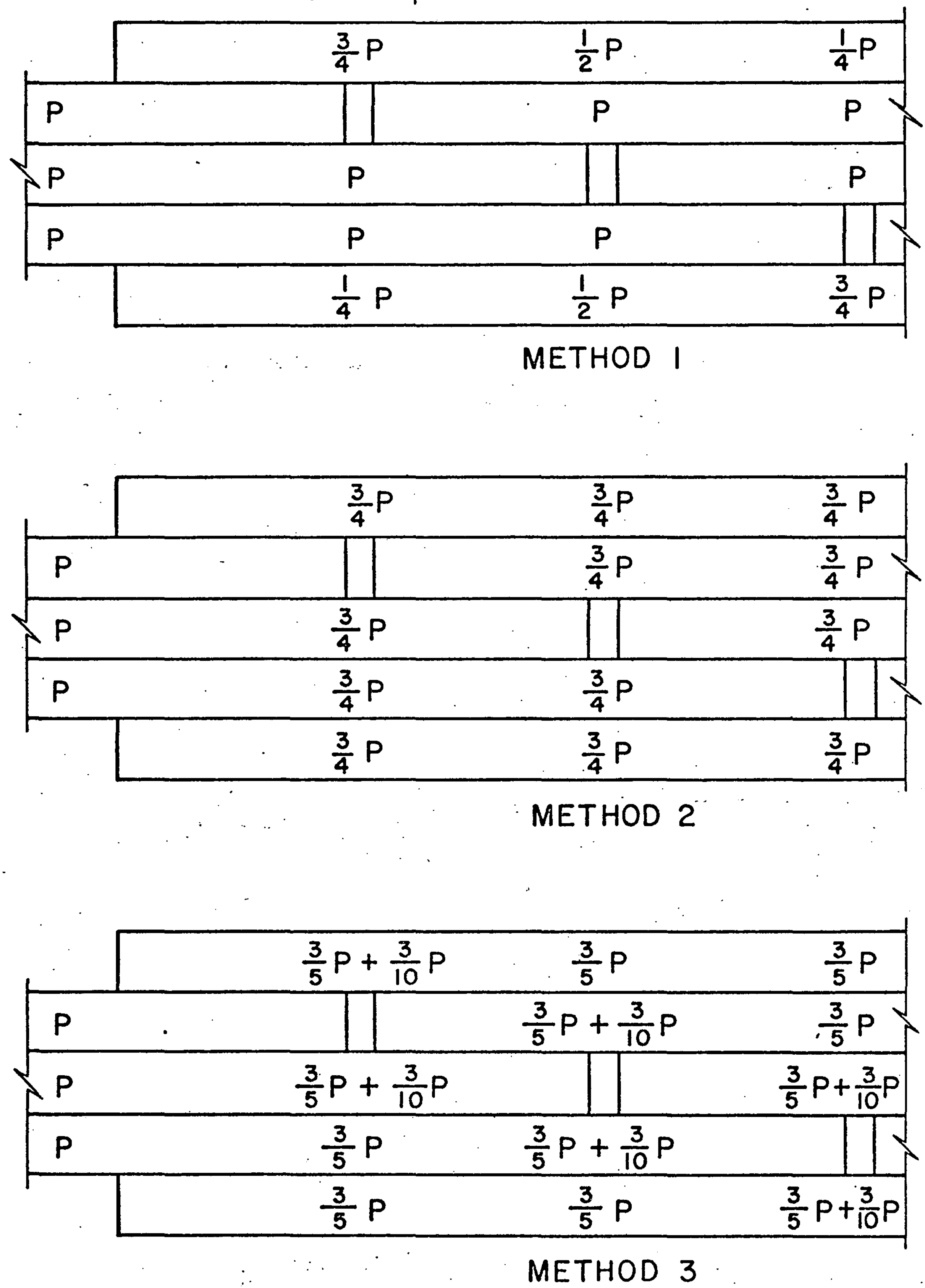


Fig. 21 Illustration of Design Methods

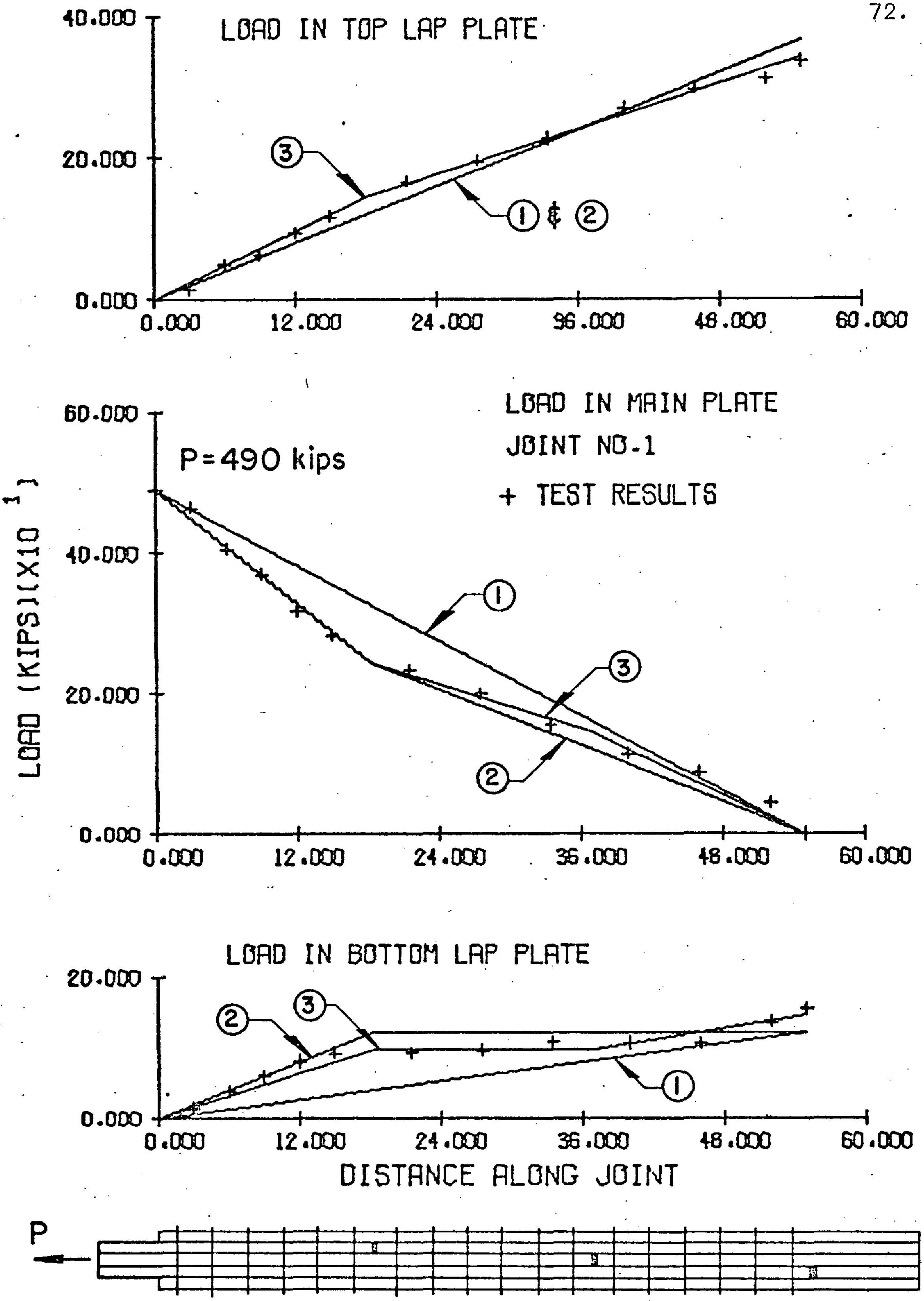


Fig. 22 Comparison of Design Methods with
Test Results of Joint 1

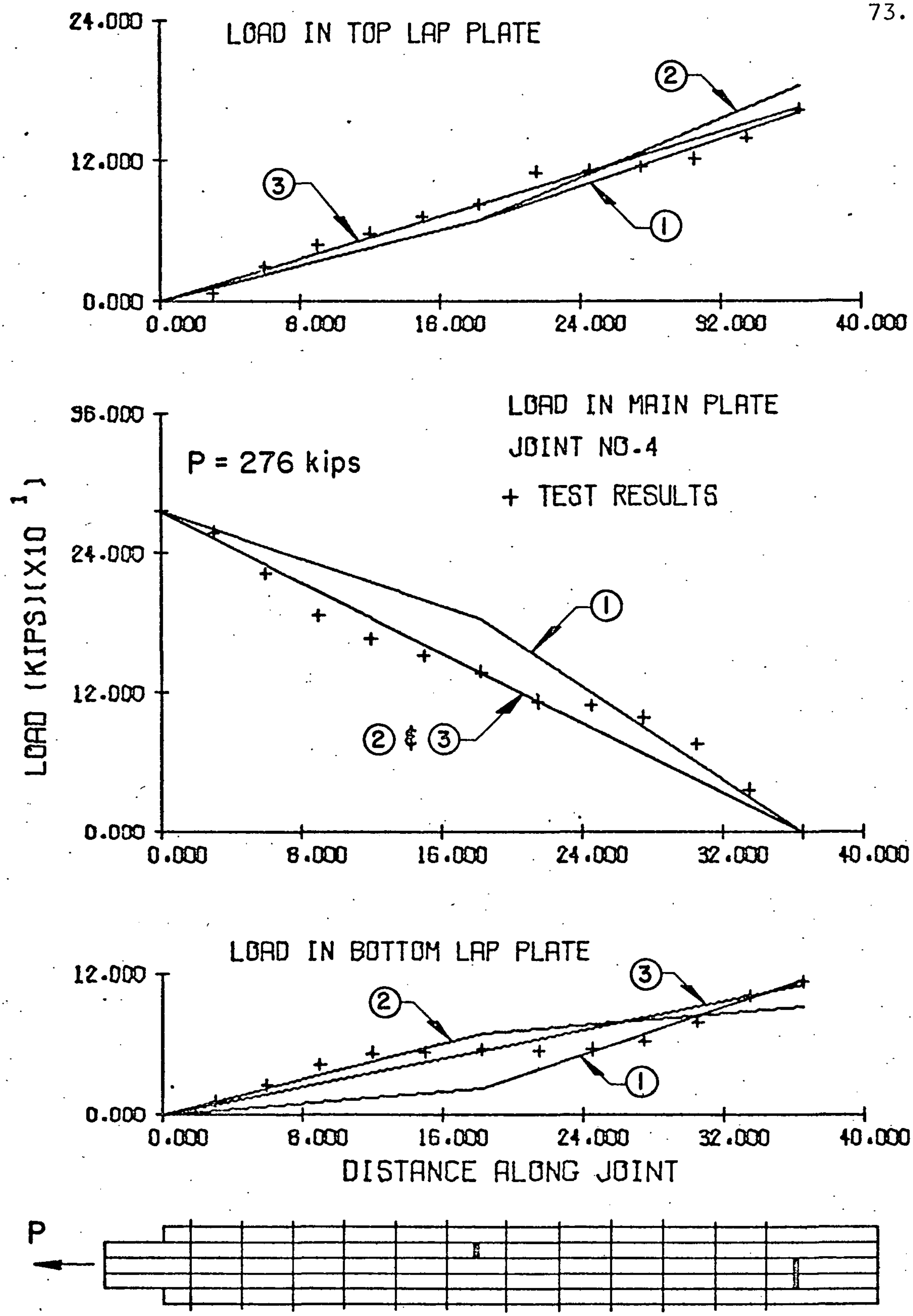


Fig. 23 Comparison of Design Methods with Test Results of Joint 4

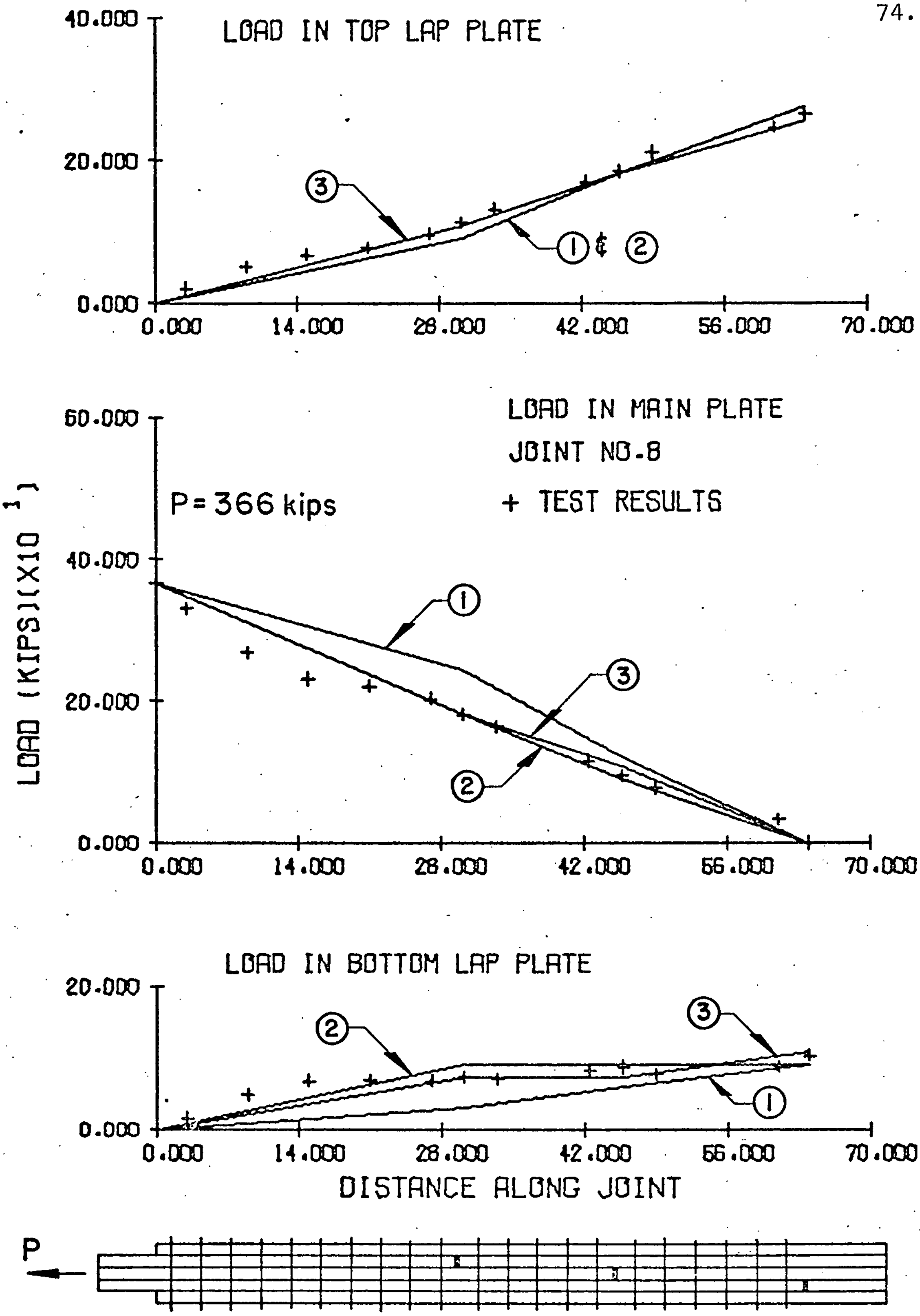


Fig. 24 Comparison of Design Methods with Test Results of Joint 8

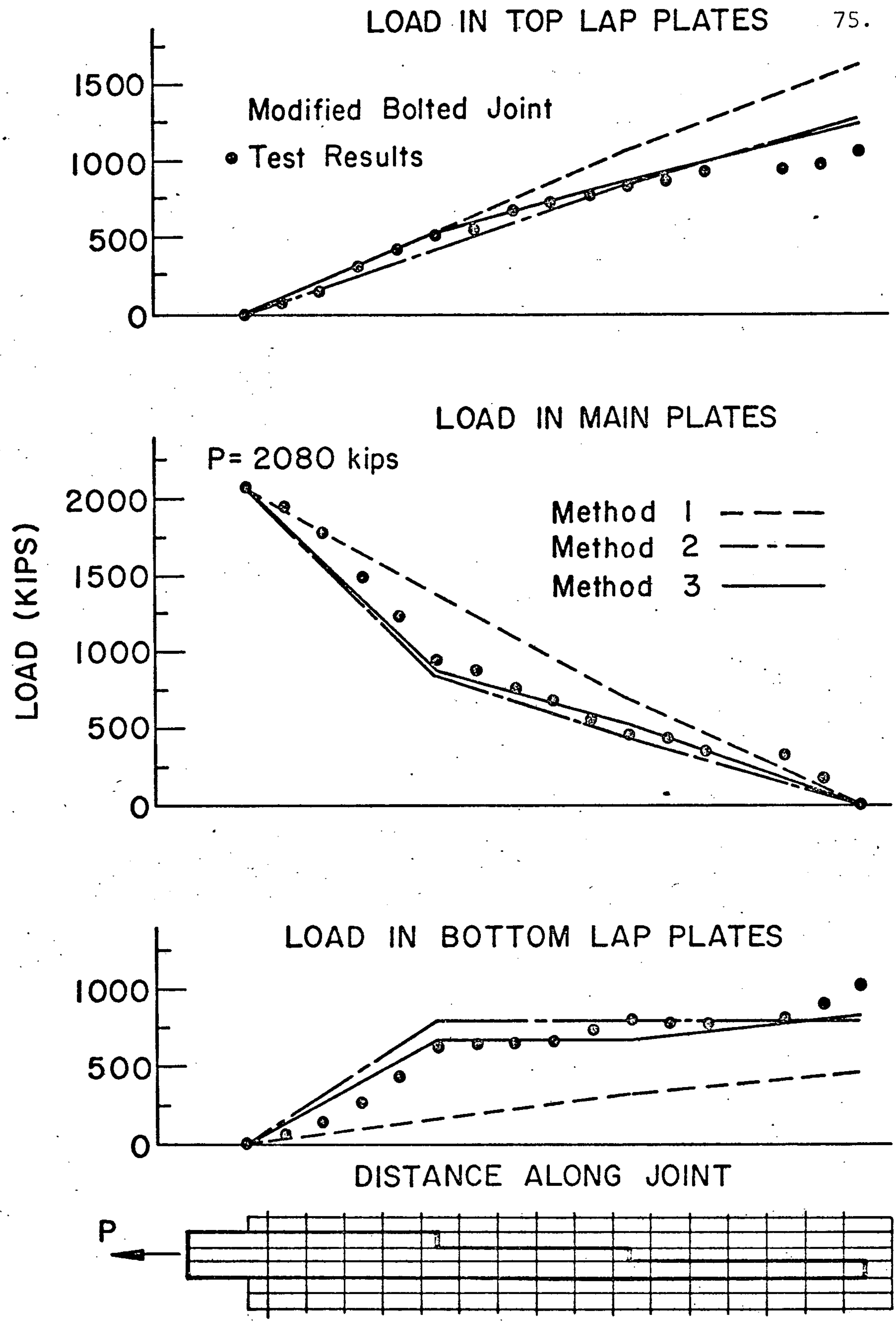


Fig. 25 Comparison of Design Methods with the Experimental Load Partition in the Modified Bolted Joint

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